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To Establish Viable Methods of Maintaining Waste Treatment Facility Efficiencies with Reference to Flow Variations

Research Report No. 11

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CANADA-ONTARIO AGREEMENT

RESEARCH REPORTS

These RESEARCH REPORTS describe the results of investigations funded under the Research Program for the Abatement of Municipal Pollution within the provisions of the Canada-Ontario Agreement on Great Lakes Water Quality. The provide a central source of information on the studies carried out in this program through in-house projects by both Environment Canada and the Ontario Ministry of the Environment, and contracts with municipalities, research institutions and industrial organizations.

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TO ESTABLISH VIABLE METHODS OF MAINTAINING WASTE TREATMENT FACILITY EFFICIENCIES WITH REFERENCE TO FLOW VARIATIONS

by

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RESEARCH PROGRAM FOR THE ABATEMENT OF MUNICIPAL POLLUTION WITHIN THE PROVISIONS OF THE CANADA-ONTARIO AGREEMENT ON GREAT LAKES WATER QUALITY

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ABSTRACT

Equalization of sewage flow variations has several potential benefits with respect to maintaining efficiencies in waste treatment facilities, the major benefits being: (i) reduced unit size requirements for treatment facilities; (ii) more stabilized process operations; and, (iii) reduced by-passing of the incoming flow. A review of the pertinent literature revealed that only very limited attention has been given to these aspects and verified the need for this project.

The first step in this evaluation was to develop a methodology for sizing equalization facilities taking into account diurnal, daily and seasonal variations in sewage flow. Actual flow data from a treatment plant having a 40-50 MIGD (million imperial gallons per day) capacity were used in development of the methodology.

The significance of flow equalization to the design of treatment facilities was developed by expanding the methodology. Revised design criteria for sizing treatment facilities were established using basic design concepts incorporating modified hydraulic characteristics of equalized sewage flow into the design where possible. The methodology was then applied to sizing treatment facilities for a plant of similar size to the selected plant operating under equalized and varying flow conditions.

Although the methodology developed was conservative in many areas, particulary in the sizing of aeration tanks and final clarifiers, it was found that a saving in capital costs would be realized by installing equalization facilities. Additional potential savings in operating costs and cost benefits attributable to improved treatment efficiencies were not included in the evaluation due to uncertainties inherent with a preliminary study of this nature.

In conclusion, the study verified the need for further investigation into the potential benefits of flow equalization. Particular emphasis should be directed towards gathering information required to more rigorously evaluate the methodologies developed for sizing equalization and sewage treatment facilities.

RESUME

La régularisation des variations du débit des eaux-vannes comporte plusieurs avantages tonchant le maintien de l'efficacité des installations de traitement des déchets, les principaux étant les suivants: (i) réduction de la taille des installations de traitement; (ii) meilleure stabilité des activités; et (iii) réduction de la déviation du débit d'entrée. Une étude bibliographique appropriée a révélé qu'on avait porté peu d'attention à ces aspects et a confirmé la nécessité du programme.

On a d'abord établi une méthodologie permettant de déterminer la taille des installation tout en tenant compte des variations diurnes, quotidiennes et saisonnières du débit. On s'est servi à cette fin des données du débit réel d'une usine de traitement ayant une capacité de 40 à 50 millions gal imp/j.

En complétant la méthodologie, on a montré le rapport entre la régularisation du débit et la conception des installations de traitement. On a établi des critères de conception révisés pour la taille des installations en se servant de conceptions fondamentales et en y ajoutant, quand c'était possible, les caractéristiques hydrauliques modifiées des débits régularisés. La méthodologie a ensuite été appliquées à la détermination de la taille des installations pour une usine de même taille que celle qui fonctionne avec des débits régularisés et variés.

Bien que la méthodologie élaborée ait été bien classique sous divers aspects, particulièrement en ce qui a trait à la taille des bassins d'aeration et des clarificateurs, on a déterminé qu'on pouvait réduire les coûts de premier investissement en installant des installations régularisées. A cause des incertitudes qui ont résulté d'une étude préliminaire de cette nature, on n'a pas inclus dans cette évaluation les économies qui pourrait être réalisées du point de vue de l'exploitation

et de la rentabilité grâce à l'amelioration du rendement.

Pour conclure, l'étude a indiqué le besoin de recherches plus poussées concernant les avantages éventuels de la régularisation du débit. Il faudrait mettre l'accent sur le rassemblement de données nécessaires pour mieux évaluer les méthodologies élaborées pour la régularisation de la taille et les installations de traitement des déchets.

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1. INTRODUCTION

The increasing public and scientific concern about the quality of our environment has stimulated efforts to reduce contributions of pollutants to the environment to an unprecedented level. Of specific interest to Canada is the changing quality of the Great Lakes system, upon which the social and economic well-being of Ontario and the nation so much depends.

The efforts can be categorized as those aimed at; reducing the discharge of untreated or partially treated wastes, providing higher removal efficiencies for specific pollutants, developing technology for the removal of pollutants not presently being removed from waste discharges, source control of pollutants and reducing the cost of existing methods of treatment. It is mainly the latter category of cost reduction, and to a lesser extent some of the other categories mentioned, which are the principal concerns of this study "To establish viable methods of maintaining waste treatment facility efficiencies with reference to waste variations".

The reason for the study stems from the basis of design, and consequently the costs, of municipal waste treatment facilities. Parameters with a major influence on cost are:

a) the mean flow and strength of the waste tributary to the facilities;

b) the variations in quantity and quality of the wastes;

c) the maximum conditions for which regulatory agencies demand a specific effluent quality.

In a given situation the values of these parameters will be defined by the nature of the sewerage system, the contribu-

tors to it, the geographical location of the system, and either the assimilative capacity of the receiving water body or the discharge regulations established by the relevant regulatory agencies. In a practical sense, it appears that significant cost reductions could be achieved by the more efficient use of those facilities which are sized to account for variations from the mean flow. Rephrased, it is practice in the design of a waste treatment facility to provide sufficient capacity for adequate treatment of the wastes at flow rates in excess of or less than the mean without the onset of detrimental conditions. The greater the variation accounted for, the less efficient is the use of the facilities.

With the increasing demand by regulatory authorities to minimize or eliminate by-passing, waste treatment facilities will have to be designed to handle even greater flow variations. However, such a requirement would result in even less efficient use of treatment facilities and more costly installations. Alternatively, the size of treatment facilities could be reduced by providing equalization facilities upstream of the plant to eliminate or minimize variations in the sewage flow.

Quality and quantity equalization of the incoming flows would enable the use of smaller facilities with possible other side benefits besides that of reducing cost, such as, improved treatment resulting in improved effluent quality, and reductions in the by-passing of untreated wastes to the receiving body. Assigning cost values to these latter benefits for accreditation to the equalized flow situation is a difficult task.

The need for the study was demonstrated by a literature review which is reported upon later, and from which it appears that the subject has been given very limited attention

to date.

In view of the foregoing, a proposal was submitted to the Canada/Ontario Agreement Research Proposals Technical Committee to study the effects of utilizing flow equalization in waste treatment facilities. The proposal was accepted and authorized under Contract Serial OGR2-0363 by the Department of Supply and Services, Government of Canada.

The following report presents the findings of the study. An initial literature review revealed that little information was available on the use of flow equalization facilities to minimize costs of municipal waste treatment or to improve the performance of municipal waste treatment facilities. Hence, a methodology based on theory was developed to compare the cost of treatment facilities with and without continuously fluctuating incoming flows. Data obtained from a full-scale facility were used where possible to develop the costs. Although it was found that the data were complete and representative of typical plant operating data, in some instances they were not extensive enough for use in the methodology. A further problem was the inability to resolve anomalies in the data which was partly expected because of the complexity of sewage treatment plant operation and the methods currently in use for reporting operating data. In such cases, data were taken from the literature on laboratory, pilot plant and full-scale studies to fulfill the requirements of the methodology. In the last resort where information from the literature was not available judgement and theoretical relationships were utilized to generate the data. A discussion is presented and conclusions are drawn, the principal ones being:

1. A methodology has been developed for sizing equalization facilities to partially or fully equalize the flow to a waste treatment plant.

2. A preliminary methodology has been developed to assess the effect of equalization on the sizing and operation of downstream facilities. Insufficient data were available from either an operating plant or the literature to apply a detailed methodology.

3. Application of the preliminary methodology, using recent techniques for the design of sewage treatment plants for the elimination of flow by-passing, demonstrated a capital cost saving by the addition of equalization facilities for a plant of 40 to 50 MGD (15,140 to 18,924 cu m/day) capacity.

4. Further elaboration of the preliminary methodology is required for satisfactory identification of the necessary information prior to full-scale or pilot plant work.

Based on these conclusions the major recommendations of the study are:

1. The current methodology should be expanded to include operating cost comparisons and the effect of partial equalization.

2. Pilot plant studies should be initiated to gain further information.

3. The application of flow equalization should be expanded into other areas.

2. LITERATURE REVIEW

A literature review was undertaken to obtain the benefit of previous works on flow equalization theory, the use of flow equalization in sewage treatment plants, and the effect of constant flow on primary and secondary treatment. The principal sanitary engineering journals, (Journ. Water Pollution Control Federation and A.S.C.E. Journ. of the Sanitary Engineering Div.) contain no references to the theory or practice of flow equalization in sewage treatment plants and articles reviewing new process developments in sewage treatment practice(6,7)do not include flow equalization.

The literature on combined sewer overflows contains references to the use of storage basins which retain the flow during peak conditions for treatment during low flow conditions. Overview papers on this aspect are available in the literature.(9,10) These papers are concerned mainly with flow control but do not discuss a constant flow to the sewage treatment plants for fixed periods of time and do not indicate the effect of flow control on the waste treatment processes.

Because of the high toxicity of certain chemical compounds to the activated sludge process, quality equalization systems are used on many industrial waste treatment schemes primarily in the refinery and petrochemical fields.(11) These basins usually are located downstream of primary treatment and upstream of biological treatment. They are constant volume basins which are designed to reduce peaks in quality not flow. A paper by Wallace(8)discusses the use of equalization basins with a constant inflow but varying quality for protection of subsequent treatment facilities. The paper presents a statistical approach to predicting the effluent quality. The mathematics of converting this model to con-

sider a varying flow entering the basin and a constant flow in the effluent would be complex.

The effect of flow variations on the operation of activated sludge plants is presented in several papers. (12, 13) However, these results are for pilot plant studies and no relationships for equalized flow have been developed.

The literature review has thus provided very little information on the use of equalization basins as a means of handling variations in flow to a sewage treatment plant.

3. THEORETICAL DEVELOPMENTS

3.1 GENERAL

Quantity and quality variations of municipal sewage occur due to the non-uniform discharge of domestic, commercial and industrial wastes according to the time of day, day of the week and season of the year. Additional variations are experienced when storm water and infiltration water enter the system, their magnitude and frequency being a function of the characteristics of the sewer system and the climatic conditions prevailing at the time. The following theoretical developments are based on equalizing defined flow variations without regard to quality. Some quality equalization will occur but insufficient information was available to determine this effect on treatment processes.

Initially, the use of an equalization basin has been considered for flow equalization. It is probable that economies could be achieved utilizing other methods of flow equalization such as routing and storage of waste flows within the sewer system but for the purposes of this report what is considered the most expensive method has been used.

3.2 SIZE OF EQUALIZATION BASIN

The primary function of the equalization basin is to temporarily store defined fluctuations in the flow entering a waste treatment plant so that a more uniform flow is fed to the waste treatment facilities. Expected advantages of equalization applied to an existing plant are improved overall treatment efficiencies, extension of the design life of the plant, reductions in operating costs and reductions in the quantities of sewage by-passing the plant. Incorporating

equalization facilities in the design of a new sewage treatment plant could result in the selection of smaller primary and secondary treatment facilities to meet design requirements in addition to those advantages mentioned above. Details of these design and performance features will be given in later sections.

Designing for maximum operating efficiency at a sewage treatment plant can be interpreted to mean that the flow would have to be regulated to achieve a constant flow throughput over the entire year. Because of the large seasonal variations of flow in Canada, a portion of the sewage from the summer months would require storage for treatment in the winter months. This was considered an impractical approach to sizing equalization facilities.

An approach considered more rational was to design the equalization basin with sufficient storage volume to eliminate diurnal variations in flow to the treatment facilities on the maximum day flow. Using a plot of data showing the frequency of occurrence of specified sewage flows in conjunction with the methodology developed for sizing the equalization basin it was possible to define the percentage of the time which a basin of a specific size was either adequate to eliminate diurnal flow variations or the percentage of the time that the full capability of the basin would be used.

The methodology developed for sizing the equalization basin was:

a) To calculate the storage volume required for elimination of diurnal flow variations in sewage flow to the treatment facilities it was necessary to know the pattern of flow variation over a day related to the daily mean flow.* Ideally,

* See Appendix A-1 for definitions of flow terminology.

the flow patterns should be evaluated for every day of recorded flow data. However, since this information is normally not available the following approach was adopted.

It was theorized that in relation to the daily mean flow the maximum diurnal variation would occur when the infiltration and storm water components of the sewage were at their minimum values. Increases in these components would in most sewerage systems, occur for longer durations than the increases caused by the non-uniform discharge of wastes from domestic, commercial and industrial users. Consequently, diurnal variations from the daily mean flow would be maximized under dry weather conditions and to be conservative the maximum variations were used to estimate the size of the equalization facility.

A typical plot of the variation in dry weather flow over a 24-hour period is shown in Figure 3.1. The plot was made statistically independent of flow by dividing the instantaneous flow rate by the daily mean flow rate. This allows a more valid comparison of several days data.

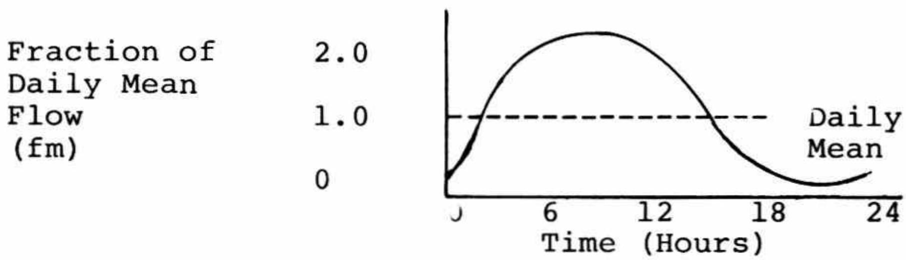


Figure 3.1 Average variation in flow over dry weather day

b) Once the variations in flow over a day have been determined as a function of the daily mean flow, the frequency of occurrence of specific daily mean flows was determined by analyzing daily mean flow values at the plant for several years of record. A typical plot of the results of the analysis is shown in Figure 3.2

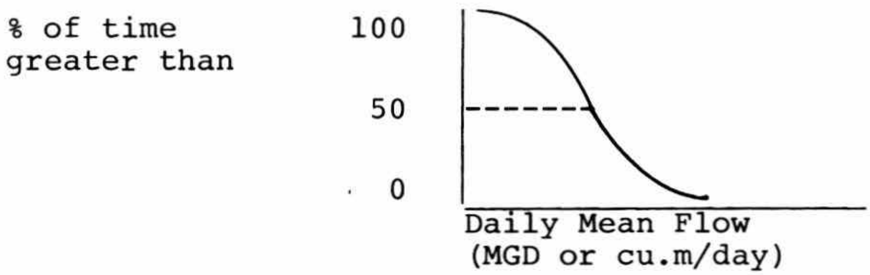


Figure 3.2 Frequency of occurrence of daily mean flows

c) With the information generated in steps a) and b) it was possible to calculate the storage volume required to eliminate diurnal variations at any daily mean flow rate. The volume required was obtained by plotting the cumulative flow over a 24-hour period and comparing the deviations of this curve with the straight line representing that uniform outflow rate from an equalization basin which would ensure that the basin was empty at the beginning of the next 24-hour period. As in step a) this can be made independent of any specific daily mean flow by plotting the hourly flow values as a fraction of the daily mean flow (obtained from Figure 3.1). By definition the cumulative sum of fm over a day must equal 24.

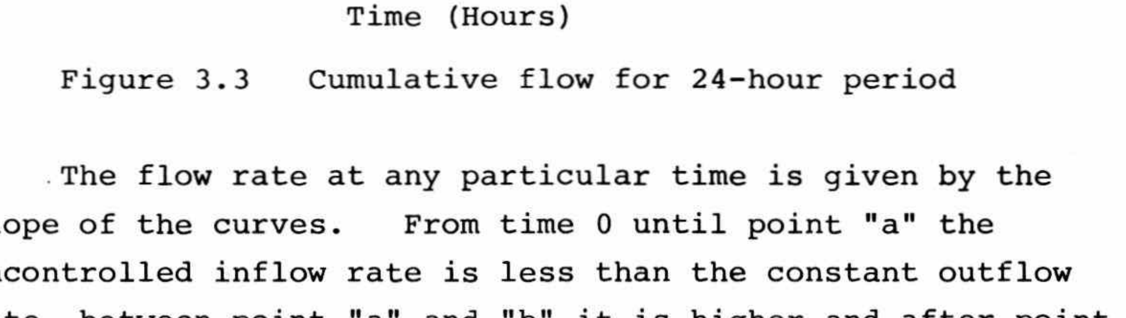


Figure 3.3 Cumulative flow for 24-hour period

The flow rate at any particular time is given by the slope of the curves. From time 0 until point "a" the uncontrolled inflow rate is less than the constant outflow rate, between point "a" and "b" it is higher and after point "b" it decreases until the basin is represented as being

empty. At points "a" and "b" the uncontrolled flow rates equal the constant rate by definition. Thus that fraction of the daily mean flow requiring storage defined here as Vm, is equal to the sum of the vertical distance between points "a" and "b" and the constant flow line as indicated on Figure 3.3.

The volume of equalization storage, V_{ES}, required for any daily mean flow, Qm, can be obtained from the following relationship.

$$V_{ES} = \frac{Vm}{24} \times \frac{Qm}{6.24} \qquad 3.1$$

where:

Vm	=	fraction of daily mean flow requiring storage
Qm	=	daily mean flow, gpd
V_{ES}	=	equalization storage volume required cu. ft.
V_{ESM}(cu.m)	=	V_{ES} (cu.ft.) x .028

d) In steps b) and c) the frequency of occurrence of any daily mean flow and the corresponding equalization storage volume required to eliminate diurnal variations at that flow were determined. Utilizing this information, a relationship for the percentage of time storage would be required if a specific equalization volume was calculated. In particular, the procedure involved selecting flows, determining their frequency of occurrence from Figure 3.2, and calculating the storage volumes required from Equation 3.1.

A representative plot of the results from the calculations is shown in Figure 3.4.

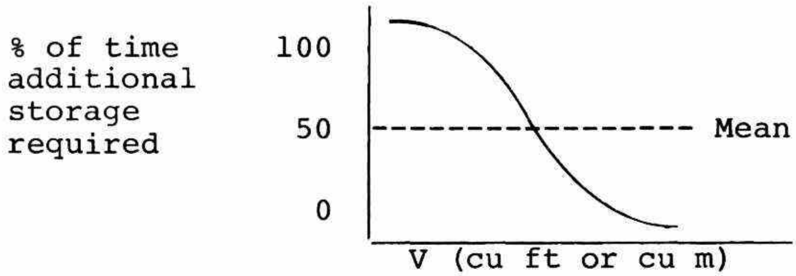


Figure 3.4 Requirement frequency for equalization volumes

From published information on costs of treatment units and using current cost indices, the cost of various sizes of equalization basins was estimated. The costs were based on constructed basins assuming average ground conditions and including adequate mixing facilities. A typical curve showing the costs is presented in Figure 3.5.

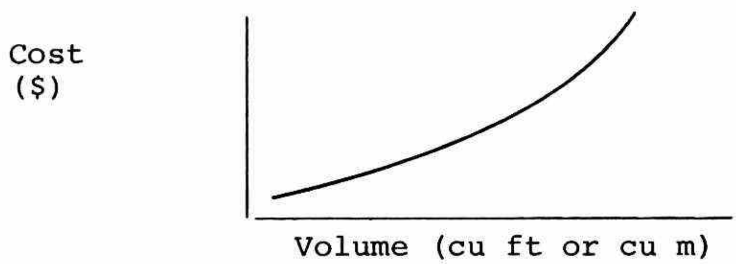


Figure 3.5 Cost for Equalization Facilities

Combining Figures 3.4 and 3.5, a relationship indicating the capital cost of constructing a storage facility and the frequency of time it would be used was developed as shown in Figure 3.6.

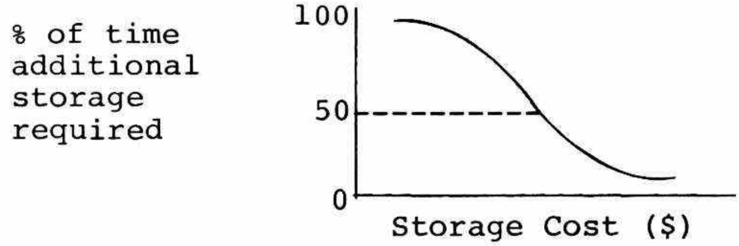


Figure 3.6 Cost versus frequency of occurrence

3.3 PRIMARY CLARIFIER DESIGN

Important parameters in the design and operation of primary clarifiers dealing with flocculent solids are the detention time, DT, and the overflow rate, OR, both of which are directly related to the design flow rate, Q, as indicated by the following equations:

$$DT = V_P/Q \qquad 3.2$$

$$OR = Q/A_P \qquad 3.3$$

where V_P and A_P are the volume and area respectively of the clarifiers. Since available information in the literature indicates that the efficiency of suspended solids or BOD removal versus overflow rate or detention time is not a linear relationship due to a dependence upon the solids particle size distribution and turbulence aspects, it is expected that a uniform flow rate to the clarifiers would improve their overall performance. The method of determining this improved performance will be outlined in this section.

The basic steps involved in designing and costing primary clarifiers subject to normal conditions of continuously fluctuating incoming flows (daily mean flow = Qm) are as follows:

a) Usually a peaking factor is selected and applied to the annual daily mean flow to calculate design flows. The factor is based on recorded and/or estimated values of the variations in flow tributary to the plant. It represents a combination of effects influencing daily variations and seasonal variations. The ASCE and WPCF Sewage Treatment Plant Design Manual (1) recommends peaking factors between two and three.

b) The removal of solids in primary clarifiers is principally related to the size distribution of the solid particles, to the settleability or rate of settling of these solids, and to the upflow velocity in the clarifiers. Removal of solids in a certain size range will be reduced to zero when the upflow velocity causes these particles to rise with the liquid phase. In general practice, the upflow velocity is measured in terms of the overflow rate. Selection of a design overflow rate for maximum flow conditions is based mainly on past experience and normally is in the range of 1,000 to 1,200 gallons/day/ft.^{2} (40-50 cu m/day/sq m).

c) The clarifier area required, A_p, is based on the maximum design flow to the plant and the selected design overflow rate. This can be calculated from the following relationship.

$$A_p = \frac{Q_m \times f_p}{OR_p} \qquad 3.4$$

d) The depth of the clarifier will be selected to be compatible with available sludge handling equipment, site conditions and experience related to expected solids removal efficiencies. With the foregoing information the cost of the clarifiers can be estimated for the case of normal fluctuating flows.

Utilization of the above approach for sizing primary clarifiers for equalized flow conditions was investigated. Considering the peaking factor, f_p, as the product of a daily variation, f_d, and a seasonal variation, f_s, leads to the conclusion that operating under equalized flows, only the factor f_s need be utilized to determine the area of clarifier required. The costs of clarifiers in the normal and flow equalized systems would then be related to $Qm \times f_s \times f_d$ and $Qm \times f_s$ respectively. The fallacy with the approach is that the different patterns of flow associated with diurnal and

seasonal variations would produce different efficiency characteristics for solids removal and the clarifiers would not be comparable.

To make the systems comparable, performance characteristics of the clarifiers must be considered. The principal parameters for measuring the performance of clarifiers are suspended solids and BOD removal. Typical operating data for a plant with fluctuating flow are presented in Figure 3.7.

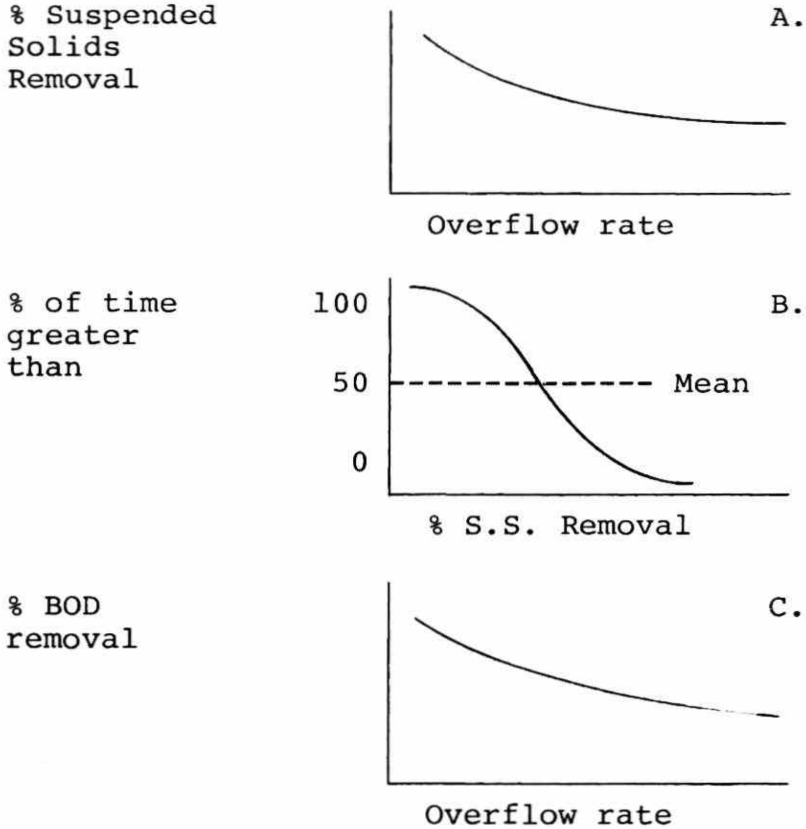


Figure 3.7 Normal operating conditions for a conventional plant

A methodology for evaluating equalized flow systems incorporating performance data was developed based on the following criteria:

a) Comparison of the cost for each type of plant based on each having the same suspended solids removal efficiency.

c) Determination of improved BOD and suspended solids removal efficiencies in equalized and fluctuating flow plants of the same size.

The methodology is as follows:

a) The relationship between the solids removal efficiency of primary clarifiers operating under fluctuating and equalized flow conditions has not been treated in the literature and therefore cannot be obtained directly. Hence two procedures for determining the relationship were identified.

The first procedure involves the use of actual plant operating data when the data has been obtained for several time intervals over 24-hour operating periods. When suspended solids removal efficiency has been measured on a sub-day basis during dry weather flow conditions at a plant receiving normal fluctuating flows a plot of solids removal versus time will vary over the day according to the typical curve shown in Figure 3.8A. Considering each of the analytical points separately, it can be argued that each data point represents the solids removal efficiency for the relatively uniform flow occurring at the time of measurement. Thus by combining the data on solids removal from Figure 3.8A with the sub-day flow data from Figure 3.1, the relationship between solids removal and overflow rate can be derived as indicated in Figure 3.8B. This procedure produces only a rough approximation of the solids removal efficiency under equalized flow conditions because the effects of detention time are not taken into consideration.

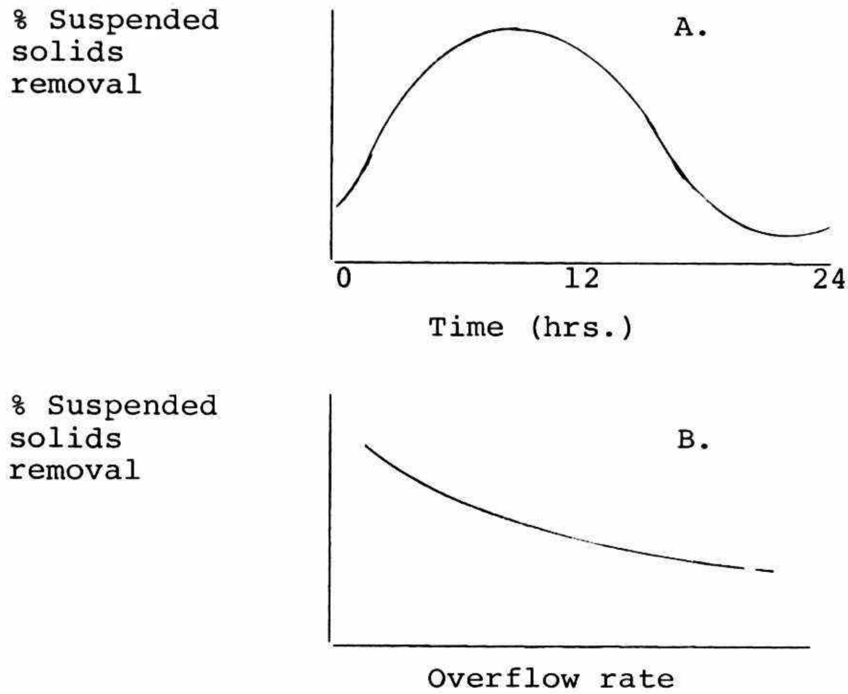


Figure 3.8 Operating conditions for equalized plant obtained from sub-day data

The second procedure involves the use of existing theories applied in the design of full scale clarifiers from batch scale sedimentation experiments. The relationships between batch experiments and varying flow reactors have been discussed in the literature, and are usually handled by a scale-up factor, f_v. This factor takes into account non-idealized conditions in the reactor such as dead spaces and corner effects as well as the varying flow conditions. Eckenfelder and O'Conner (2) have indicated that f_v is in the range of 1.75 to 2.0.

The effects of non-idealized conditions in primary clarifiers have been investigated by several researchers utilizing tracer study methods. Wallace(3)has calculated that the non-idealized factor, f_n is approximately 1.2. The effects of non-idealized conditions and varying flows on clarifier performance are multiplicative rather than additive. Thus the factor applicable to varying flow only, f_e, can be determined

from the relationship:

$$f_e = \frac{f_v}{f_n} \simeq \frac{1.75}{1.2} = 1.45 \qquad 3.5$$

Since non-idealized conditions will influence the performance of clarifiers operating under either fluctuating or equalized flow conditions, the principal difference in performance of the two systems is related to the varying flow effect. An approximation of the expected performance of a system operating under equalized flow conditions therefore, can be derived by applying the factor, f_e, for varying flow to actual daily operating data of a plant operating under normal fluctuating flow conditions. That is, an approach which has been used to obtain fluctuating flow data from batch operating data can also be used to obtain equalized flow data from varying flow data as shown in Figure 3.9.

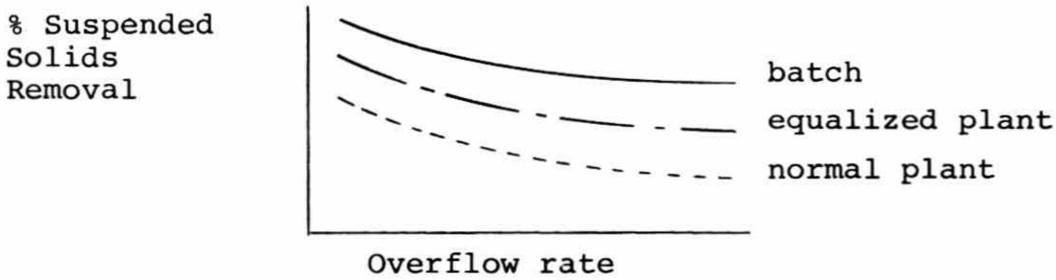


Figure 3.9 Operating conditions for equalized plant from daily data

b) To apply criteria "a" as a basis of comparing the plants, both the fluctuating and equalized flow systems must have the same mean annual suspended solids removal efficiency. The relationship between solids removal efficiency and overflow rate has been defined in Figure 3.9. Thus for a constant size clarifier the solids removal efficiency varies over the year with the fluctuation in flow.

Although the equalized flow plant will be designed to provide 100 percent equalization to the maximum daily flow, the plant will still receive seasonal variations in flow. However, the equalization facilities will provide the flexibility of maintaining a constant flow to the plant for periods of several days as indicated by the step-function curve in Figure 3.10.

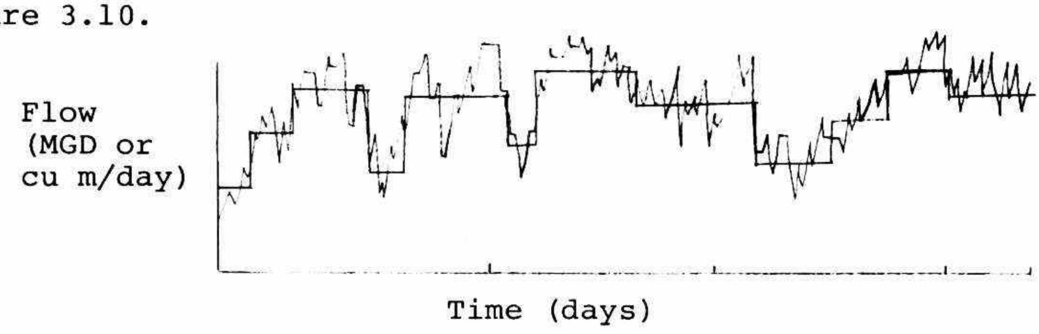


Figure 3.10 Selection of Constant Flow Periods

To conveniently handle the data, the following grouping of flows could be used:

a) daily flow
b) weekly average flow
c) two week average flow

Determination of the primary clarifier surface area required for the equalized flow system to achieve the same mean annual solids removal efficiency as the varying flow system must be done by trial and error. An area is selected and then overflow rates are calculated for each of the flow groupings. The suspended solids removal efficiency associated with each overflow rate is calculated from Figure 3.8 or 3.9. The frequency of occurrence of the solids removal efficiencies are calculated to determine the mean annual solids removal efficiency. If the mean calculated solids removal efficiency differs substantially from that for the varying flow system, a new surface area is selected and solids removal efficiencies are again calculated. Following this procedure,

solids removal frequency curves similar to those shown in Figure 3.11 are generated.

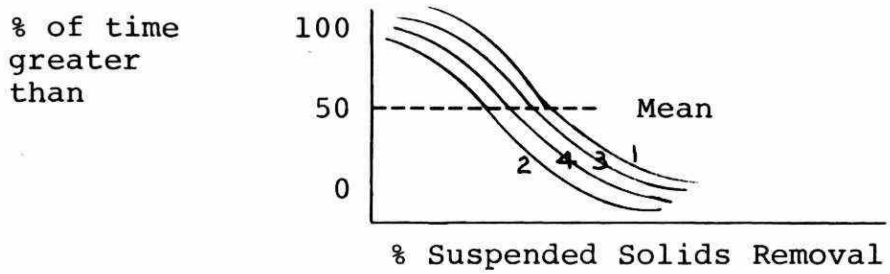


Figure 3.11 Calculation of Primary Clarifier Area Under Equalized Flow

The procedure is repeated with new trial surface area values until the mean solids removal efficiency for the equalized flow system is comparable to that for the varying flow system. The final area is really only applicable to the grouping of flows used in the calculations. The effect of these various groupings can be determined by comparing the results of the areas calculated for each of the groupings listed above.

Once the area required for the primary clarifier has been determined, a final cost can be estimated.

c) To determine the improvement in suspended solids removal efficiency which would be realized by adding equalization facilities to an existing plant, the same methodology outlined in step b) above is used. In this case, however, the area is known and the mean efficiency of solids removal is calculated directly for each flow grouping. Again, the effect of various flow groupings can be determined by comparing mean values and surface area requirements.

The relationship between BOD removal efficiency and overflow rate also can be evaluated from sub-daily data or from scale-up factors as was done above for suspended solids

removal. The use of sub-day information does not produce accurate results and essentially no work has been recorded in the literature on the scale-up relationships. Thus a BOD comparison is not possible at this time.

3.4 AERATION TANK DESIGN

Current concepts in the design and operation of activated sludge processes indicate that the ratio of the organic loading to the biological mass carried in the aeration system is the most important parameter. In general, this relationship is approximated by the ratio of the daily BOD loading, applied to the aeration tanks, to the total weight of volatile suspended solids maintained under aeration. The ratio is important in determining the BOD removal efficiency of the process and the settling characteristics of the mixed liquor solids.

Other criteria of major importance to the designers of activated sludge plants can be summarized as follows:

1. The BOD loading per unit volume of aeration tank
2. The detention time in aeration
3. The mixed liquor suspended solids concentration
4. The quantity of air/oxygen required
5. Settling characteristics of the mixed liquor solids
6. Character and quantity of returned sludge

In general practice, biological units are designed conservatively to minimize adverse effects which variations in the quality and quantity of the waste water can have on the process. Although operating under equalized flow conditions eliminates variations in flow over a day, it only partially equalizes quality variations. Equalization does, however, minimize shock loadings through mixing of different segments

of the flow and thus provides a more uniform feed of organic material to the process.

The steps normally followed in the design of a biological treatment plant receiving normal fluctuating flows as defined in the ASCE and WPCF Sewage Treatment Design Manual(1) are as follows:

a) Select daily organic loading rate (F/M).

The organic loading on the activated sludge process is defined as

$$\frac{F}{M} = \frac{BOD_{IN}}{MLVSS \times DT} \tag{3.6}$$

where BOD_{IN} and MLVSS are respectively the biological oxygen demand of organic load entering the biological unit and the mixed liquor volatile suspended solids in the unit. DT is the detention time. In practice the total mixed liquor suspended solids, MLSS, is frequently used instead of the MLVSS. Operating experience has indicated that a poor settling sludge and deterioration of treatment may occur when the loading rate is greater than 0.5 lb. BOD/lb. MLVSS/day (kg BOD/kg MLVSS/ day). When the loading rate falls below 0.25, nitrification also takes place increasing the overall air requirements. A loading of 0.3 to 0.35 is usually selected for conventional activated sludge processes receiving normal fluctuating flows.

b) Determine mean BOD of material entering biological plant.

Frequently, the characteristics of a particular sewage are not defined and a BOD concentration must be assumed. Also, it is generally necessary to estimate the reduction in BOD which will occur in primary treatment when these facilities are included as part of the design. Curves giving the degree

of BOD removal in primary tanks for various detention times are given in Fair and Geyer.(4) In normal size plants a BOD reduction of 25 to 30 percent is achieved.

c) Select optimum mixed liquor volatile suspended solids level.

Optimum efficiency in the biological process is normally obtainable when the MLVSS concentration ranges between 1,200 and 2,500 mg/l with 1,500 to 1,800 mg/l being optimum.

d) Determine aeration tank volume.

The volume of the aeration tank required, V_A, can be calculated from equation 3.7 as follows:

$$V_A = \frac{BOD_{IN}\ Qm}{MLVSS\ (F/M)} \qquad 3.7$$

The method described above is based on extensive operating experience that has resulted in the selection of a loading rate of about 30 lbs. of BOD per 1,000 cu. ft. (.5 kg/cu m) of tank volume as a conservative design figure for an activated sludge plant. As this design criterion is empirical, based on operating data from plants having varying flow, it is not correlatable with parameters which can be used to determine aeration volume requirements for equalized flow conditions. To overcome this difficulty an alternative method of determining the volume of an aeration basin required for varying flow conditions is proposed. The method is based on parameters which are flow dependent. The steps involved in this method are as follows.

a*) Select BOD_{IN}, MLVSS and the rate of return sludge, R_m.

This can be done by the methods outlined above in steps b, c, and e respectively.

b*) Determine BOD_{OUT} to be maintained.

The limits on BOD in the plant outlet are generally set by the regulatory authority for that area.

c*) Determine the first order BOD removal rate constant, k.

The rate of BOD removal in a completely mixed biological system is given by the relationship

$$\frac{BOD_{OUT}}{BOD_{IN}} = \frac{1}{1 + k.MLVSS.DT} \qquad 3.8$$

The rate constant, k ($days^{-1}$) is dependent on the sewage characteristics, the biological process involved and the temperature of the waste. Thus k must be evaluated on a simulation of the system to be employed. It is preferable that this be done by pilot plant studies in the case of new plants or by using actual operating data in the case of existing plants.

d*) Determine aeration tank volume and cost.

Once a k value has been evaluated, equation 3.8 can be solved to determine the detention time, DT. The aeration volume can then be obtained from the relationship

$$V_A = DT\ (Qm + Rm) \qquad 3.9$$

Application of the above approach is dependent on being able to determine the BOD removal rate constant and effluent BOD from the biological process. Typically, plant operating

data are available on only the effluent after final clarification and for only the total BOD. The contribution of solids in the final effluent to the BOD value usually is not measured, therefore, it is not possible to obtain a true measure of the BOD removal rate constant. Calculation of the k rate based on total BOD measurements offers a feasible solution when due regard is given to influence of effluent solids. Typical plant operating data will yield curves similar to those shown in Figure 3.12. The curves reflect the variations occurring in a plant receiving normal fluctuating flows.

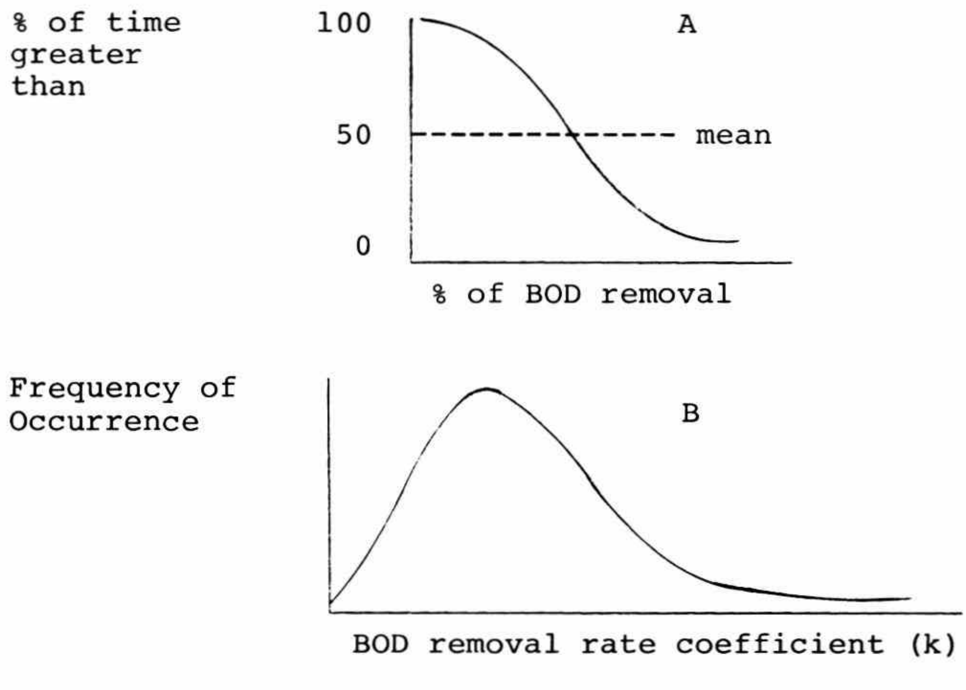


Figure 3.12 Normal operation conditions for a biological treatment plant

e) Determine air requirements.

Sizing of the air supply equipment is commonly based on the "Ten State Standards" (1) approach. This approach defines normal air requirements as 1,000 cubic feet of air per pound of BOD removed per day (62 cu. m. air/kg BOD) and specifies

that the air supply system be designed so as to be capable of delivering 150 percent of normal requirements. Based on past operating experience with diffused air aeration systems, it was assumed this would provide sufficient oxygen to meet the requirements for oxidation of organic matter and metabolism of cell mass.

A more detailed approach of sizing diffusion equipment, defined by Equation 3.10, encompasses oxygen transfer efficiencies and waste characteristics

$$O_s = O_a \times \frac{9.2}{C_B \times P \times \beta - DO} \times \frac{1}{\alpha} \times \frac{1}{1.024^{(T-20)}} \qquad 3.10$$

where

O_s = standard oxygen requirement (lb. O_2/hr. or kg O_2/hr.). Standard operating conditions being defined as clean water, 20°C operating temperature, 1 atmosphere dry pressure and a DO residual of 0 mg/l.

O_a = actual oxygen requirement (lb. O_2/hr. or kg O_2/hr.)

α = ratio of the oxygen transfer coefficient (K_La) of waste to that of clean water.

β = ratio of oxygen saturation in waste to that of clean water.

C_B = saturation of oxygen in water at operating temperature and 1 atmosphere dry pressure (mg/l).

9.2 = saturation of oxygen in water at standard conditions. (mg/l)

T = temperature of the mixed liquor in the aeration basin (°C).

P = atmospheric pressure at treatment plant location.

DO = dissolved oxygen level maintained in the aeration basin (mg/l).

The parameters for any waste are determined from pilot plant testing or experience. For municipal wastes the following parameters are commonly used; $\alpha = 0.9$, $\beta = 0.9$, DO = 2 mg/l, a temperature of 15 to 17°C in the reaction tank, with O_a based on 1 pound of oxygen per pound of BOD removed (1 kg O_2 /kg BOD removed). For commonly used diffuser equipment this results in air requirements similar to those calculated by the Ten State Standard approach for normal operating conditions, i.e. 1,000 cu. ft. of air/lb. BOD (62 cu. m. air/kg BOD). This air requirement must also be increased by 50 percent to provide additional oxygen during periods of peak demand.

With either of the above approaches, the air supply system is designed to provide 150 percent of the normal operating requirements based on average BOD loadings. Design on this basis provides standby capacity under normal operating conditions and a built-in capacity for supplying the biological process with additional air under peak loading conditions.

The rationale for providing 50 percent excess air capacity is based on past operating experience and not a rigorous design basis. In view of current trends to minimize or eliminate by-passing, thus providing complete treatment to all of the waste, coupled with demands to minimize odours from

sewage treatment plants, it is paramount that adequate air must be supplied to the biological process under all conditions of operation. A more rigorous basis for sizing air equipment is therefore required.

As the oxygen requirements of the biological process are directly related to the BOD loading on the process, the sizing of the air supply system should involve the peak BOD loading. Information relating the occurrence of peaks in BOD to hydraulic peaks are generally lacking, however, because of the flushing action in the sewers it is expected that the peak BOD loading will occur with the peak flow to the plant.

An initial approach is to use the average BOD concentration coupled with a hydraulic peaking factor. Such a factor was developed for a recent design project by the writer's firm and it is felt that this approach will be used increasingly in the future. The hydraulic peaking factor developed was

$$f'_a = \frac{\text{maximum 12 hour flow rate}}{\text{mean daily flow rate}} \qquad 3.11$$

which can also be expressed as

$$f_a = \frac{\text{maximum 12 hour flow on mean day}}{\text{mean daily flow rate}} \times \frac{Qmax}{Qm} \qquad 3.12$$

The capacity of the air equipment, S_a, in CFM, can thus be calculated from

$$S_a = f'_a\ Qm(\text{MGD}) \times 10(\text{lb./gal}) \times BOD_R\ (\text{mg/litre}) \times \frac{1000\ (\text{ft}^3/\text{lb.})}{1440\ (\text{min/day})}$$

$$= 6.95\ f_a Qmax\ BOD_R \qquad 3.13$$

$$Sam(\text{cu m/min}) = Sa(\text{CFM}) \times .0283$$

where BOD_R is the mean BOD removed in the aeration tank. This design is somewhat conservative in that average BOD concentration and a 12 hour peaking factor were used.

f) Determine return sludge requirements.

The rate of sludge return is dependent on the amount of suspended solids to be maintained in the aeration tanks and the concentration of solids in the return sludge. Formerly plants were designed with return sludge capacities of 10 to 30 percent of the average sewage flow. Newer plants are designed to provide return capacities of 70 to 100 percent of the design flow. The reason for the increase in return sludge capacity has resulted from the realization that maintaining the MLVSS in the optimum operating range is critical to the performance of the biological process.

A more rational basis of determining the maximum rate of sludge return, Rmax, on the basis of a solids balance on the biological unit is as defined in equation 3.8. To calculate the maximum rate of return, assumptions must be made of the concentration of solids in the return sludge, C_S, and of the hydraulic peaking factor on the biological unit, f_b. The return sludge solids concentration is generally in the order of 0.8 to 1.2 percent solids by weight.

$$R_{MAX} = f_b \; Qmax \quad \frac{MLVSS}{C_S - MLVSS} \qquad 3.14$$

The hydraulic peaking factor, f_b, applied to the biological process is usually less than the factor, f_p, applied to the primary treatment unit. The reason for selecting a lower factor is related to the sensitivity of the biological process. Because the biological process is more sensitive than the primary treatment process, peak hydraulic loadings in excess of pre-selected values are by-passed around secon-

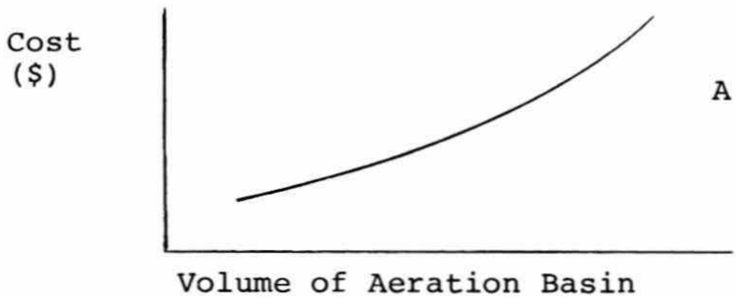
dary treatment. As not all of the sewage receives secondary treatment by design, the peaking factor is accordingly reduced. With increasing demands to minimize by-passing, the peaking factor to be applied to biological process design in practice is undergoing considerable review and in general is increasing.

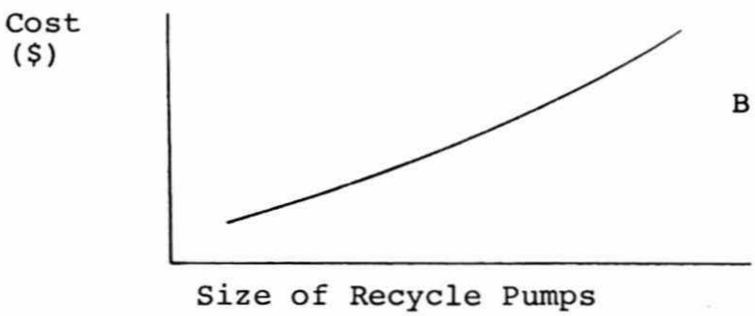
$$f_b = \frac{\text{selected peak flow rate}}{\text{mean flow rate}} \qquad 3.15$$

For the purposes of this report a maximum 6 hour peaking factor was selected to correspond with the detention time in the aeration basin.

g) Cost of aeration plant.

The total capital cost of an aeration plant can be obtained from curves similar to those in Figure 3.13 by summing the cost of the individual components.





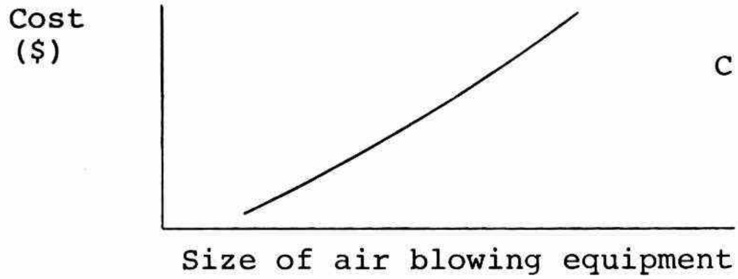


Figure 3.13 Cost of Aeration Plant

As noted previously, in the section on primary treatment, a common basis for comparing equalized and varying flow plants must be established in order to determine the size of facilities and associated costs for a plant receiving equalized flow. The criteria selected for comparison are:

(a) Comparison of the cost for each type of plant based on each having the same BOD removal efficiency.

(b) Determination of improved BOD removal efficiency in an equalized plant compared to a normal plant of the same size.

Both criteria neglect the effect which final clarification has on BOD removal. Further, the possibility of a lower BOD concentration in the feed to the equalized flow plant has not been considered.

For conditions of 100 percent equalization of the maximum daily flow, the cost of the biological process can be determined as follows.

a) Aeration Basin Volume

In order to size an aeration basin for a plant operating under equalized flow conditions so that the plant attains the

same BOD removal efficiency as a plant operating under varying flow conditions, it is necessary to define the value of the BOD removal rate constant. The value of the constant, k, is known to be dependent upon the transient conditions in the biological process and must be adapted to constant flow conditions.

Information on the effect of transient and steady state conditions on k values are available in the literature. This information, however, is primarily based on laboratory studies with synthetic wastes and is not directly applicable to large plants. In addition, the steady state conditions are based on a constant quality feed, which is not always achieved under equalized flow conditions.

The use of sub-day data from plants receiving varying flow to represent equalized conditions also is not applicable for determining the k value. Variations in the feed quality for uncontrolled flows will be much greater than under equalized flow conditions and thus values of k determined from sub-day data would not be representative of the k value for an equalized flow plant.

Further research is required to obtain the relationship for k between equalized flow and normal varying flow conditions. Current information tends to indicate that improved performance will be obtained under equalized flow conditions, or alternatively that the same performance will be obtained with a smaller reactor volume. However, due to the lack of any concrete information at this time it is assumed that the reactor volume will remain the same (criterion "a") and no significant operational improvement will be realized (criterion "b").

b) Aeration Equipment

The conventional methods of sizing air equipment, using empirical factors and average flow conditions, are based on previous plant experience with varying flow conditions. Because flow patterns differ in an equalized flow plant, the conventional methods of sizing air equipment are not directly applicable. However, if the new design techniques, incorporating a flow peaking factor to prevent flow by-passing and odour problems are used, cost savings will result for the equalizing flow case. Under equalized flow conditions the 12 hour maximum daily peaking factor can be eliminated from equation 3.16 because the plant will receive a uniform flow over the day.

$$S_s \text{ (equal)} = 6.95\ Q_{MAX}\ BOD_{IN} \text{ (cu ft)}$$
$$S_{am} \text{ (equal)} = S_a \text{(equal)} \times .0283 \text{ (cu m)} \qquad 3.16$$

c) Sludge Recycle Equipment

The conventional techniques used to design return sludge pumping capacity based on 70 to 100 percent of the mean daily flow cannot be related to equalized flow conditions. However, using the rational approach previously discussed, which is based on a solids balance, results in the following equation for calculating return sludge capacity under equalized flow conditions.

$$R_{MAX} \text{(equal)} = Q_{MAX} \frac{MLVSS}{C_S - MLVSS} \qquad 3.17$$

Thus the reduction in capacity of sludge recycle pumping equipment is proportional to the peaking factor f_b.

d) Sludge treatment

Sludge treatment facilities in an activated sludge plant are generally based on about one month storage capacity. Compared with this long detention time in the digestion facilities equalization of daily flows will have an insignificant effect on the total capacity required. Thus it is assumed that no cost savings would be realized for sludge treating facilities.

3.5 FINAL CLARIFIER DESIGN

In a previous section on primary clarifier design, the principles behind the design and operation of settling basins were briefly discussed. Because solid material carried in raw sewage streams usually has a wide distribution in size and density, the solids settle as discrete particles and removal of these in primary clarifiers is classified as unhindered settling. In contrast, biological solids have a narrow size and density range and settle at a fairly uniform rate. As a result, the particles interfere with the motion of other particles effectively reducing the settling velocity. This type of settling is termed hindered settling. Under hindered conditions the particles can also form into a zone and from then on settle collectively at a reduced rate. A distinct interface will thus be formed between the subsiding particles and the clarified liquid which is commonly termed zone settling. Most final clarifiers operate under zone settling conditions.

The operation of primary clarifiers in sewage treatment plants having secondary treatment facilities is not considered critical to the overall plant performance as solids not removed in the primary tanks are removed in the secondary facilities. On the other hand, final clarifiers must be essential-

ly fail safe as far as solids and BOD removal is concerned as any material not removed is discharged directly to the receiving waters. For these reasons the design and performance of final clarifiers is critical to the overall plant operations.

An outline of the steps followed in final clarifier design for varying flow conditions is as follows.

a) Select peaking factor

The final clarifiers must be designed to handle all the flow treated in the activated sludge process, thus the peaking factor for the final clarifier, f_c, must be based on the maximum hour peak flow.

b) Determine design maximum overflow rate, OR_F

As the successful removal of solids in final clarifiers is essential in order to meet effluent regulations, the selection of the maximum design overflow rate is most important and should be based on pilot plant studies when possible. Under zone settling conditions there exists a limiting upward liquid velocity (i.e. overflow rate) above which solids will not settle but will be carried over the effluent weirs. Current design practice is to maintain an overflow rate well below this maximum value and to choose a maximum design overflow rate for peak flow conditions somewhat below the actual maximum. When pilot plant studies are unavailable, a maximum design overflow rate of 800 gallons/day/sq. ft. (32 cu m/day/sq m) is usually used for domestic wastes.

c) Calculate tank area required, A_F

The clarifier surface area required, A_F, is calculated using the maximum peak flow and the design overflow rate.

$$A_F = \frac{f_c \; Qmax}{OR_F} \qquad 3.18$$

d) Determine cost of final clarifier

The physical structure of primary and final clarifiers is essentially the same, thus the same cost curve can be used for both the primary and final clarifiers.

Typical operating curves for final clarifiers treating varying flows are presented in Figure 3.14. Although the primary function of final clarifiers is to remove suspended solids, the concentration of BOD in the final effluent is also an important consideration. As indicated by these curves, there is a rapid deterioration in the performance of final clarifiers beyond some maximum overflow rate.

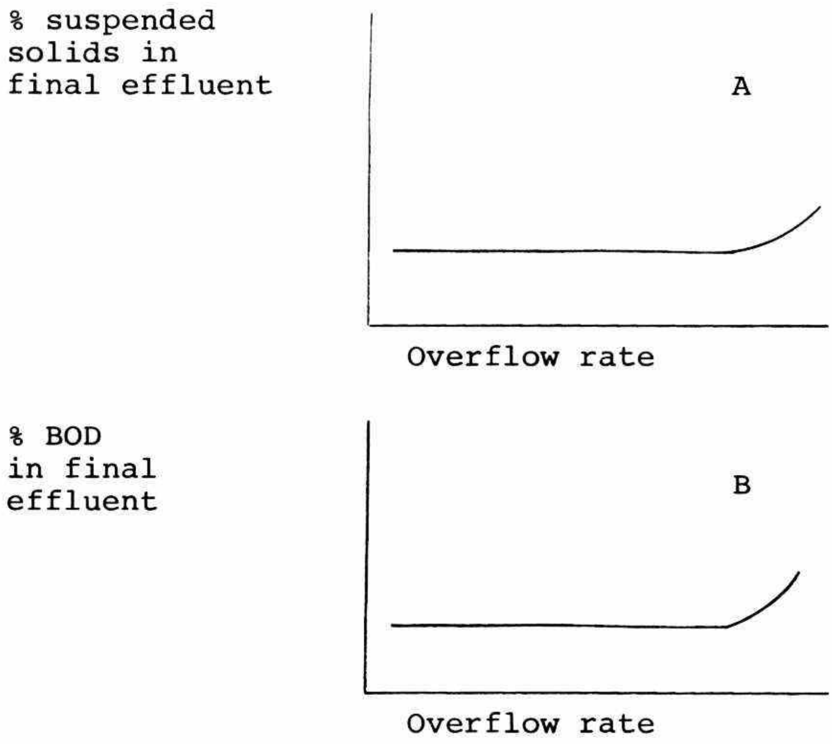


Figure 3.14 Operating Characteristics of final clarifiers

In selecting criteria to standardize the design bases of final clarifiers for equalized and varying flow conditions, it was established that the criteria cannot be based on evaluation of improved unit operations as was done for other units. As the final clarifiers are essential to overall plant performance, the design must be based on some maximum upflow velocity for both flow conditions. Any cost saving realized in meeting this criteria for the equalized flow case therefore is the basis for comparison.

The sizing of clarifiers for conditions of varying flow is based on the maximum hourly flow and a maximum design overflow rate. Under equalized flow conditions the maximum flow to the final clarifier will be the maximum daily equalized flow, Q_{ME}, as opposed to the peak hour flow on the maximum day used to determine the clarifier surface area for varying flow conditions.

The design overflow rate for final clarifiers under varying flow conditions is based on a maximum overflow rate of about 800 gal/day/sq ft (32 cu m/day/sq m) with an average operating overflow rate of about 400 gal/day/sq ft (16 cu m/day/sq m). Under varying flow conditions peak flows occur only for short periods during the day thus the overflow rate is exceeded for only short durations. With equalized flow conditions if the overall rate exceeds the design overflow rate it will do so for the entire period of continuous flow, i.e. a day or more. This would be a very critical situation thus a more restrictive overflow rate is needed under equalized flow conditions. Under normal flow conditions the average overflow rate for varying conditions of about 400 gal/day/sq ft (16 cu m/day/sq m) should be maintained, thus a suggested design overflow rate for equalized flow OR_{FE} would be 600 gal/day/sq ft (24 cu m/day/sq m). The area of the final clarifier under equalized flow can thus be calculated from

$$A_F(\text{equal}) = \frac{Qm}{OR_{FE}} \qquad 3.19$$

3.6 COST COMPARISON OF EQUALIZED FLOW AND VARYING FLOW PLANTS

The advantages of operating under equalized flow conditions have been detailed in the previous sections. The cost of equalization facilities has been detailed in Section 3.2 while the cost savings of operating a varying flow plant under equalized flow conditions have been described in Sections 3.3, 3.4 and 3.5. A comparison of the cost savings with the cost of equalization facilities gives an indication of the relative expense or savings using equalized flow. It must be stressed, however, that these costs are based on standard curves and are not exact. In addition these curves do not necessarily consider the proper number and sizes of pumps and other auxiliary equipment. Standard curves are used so that the equalized and fluctuating flow plants can be compared on the same basis.

4. EVALUATION OF METHODOLOGY

Operating data from a sewage treatment plant utilizing primary and secondary treatment were necessary for the evaluation of the methodology developed in Section 3. A general schematic of the plant selected, designated as Plant "A", is described in Appendix A-3. A description of the plant flows and recycle streams and the location of the measurement points for the recorded data are presented on the schematic.

Plant "A" was chosen for the initial evaluation of the methodology because of: the treatment processes employed; the minimum number of recycle streams which tend to simplify the interpretation of operating results and extensive daily data as well as some hourly data which were available.

The calculations involved in evaluating equalization facilities for this plant using the theoretical developments from the previous section are outlined in the following sections.

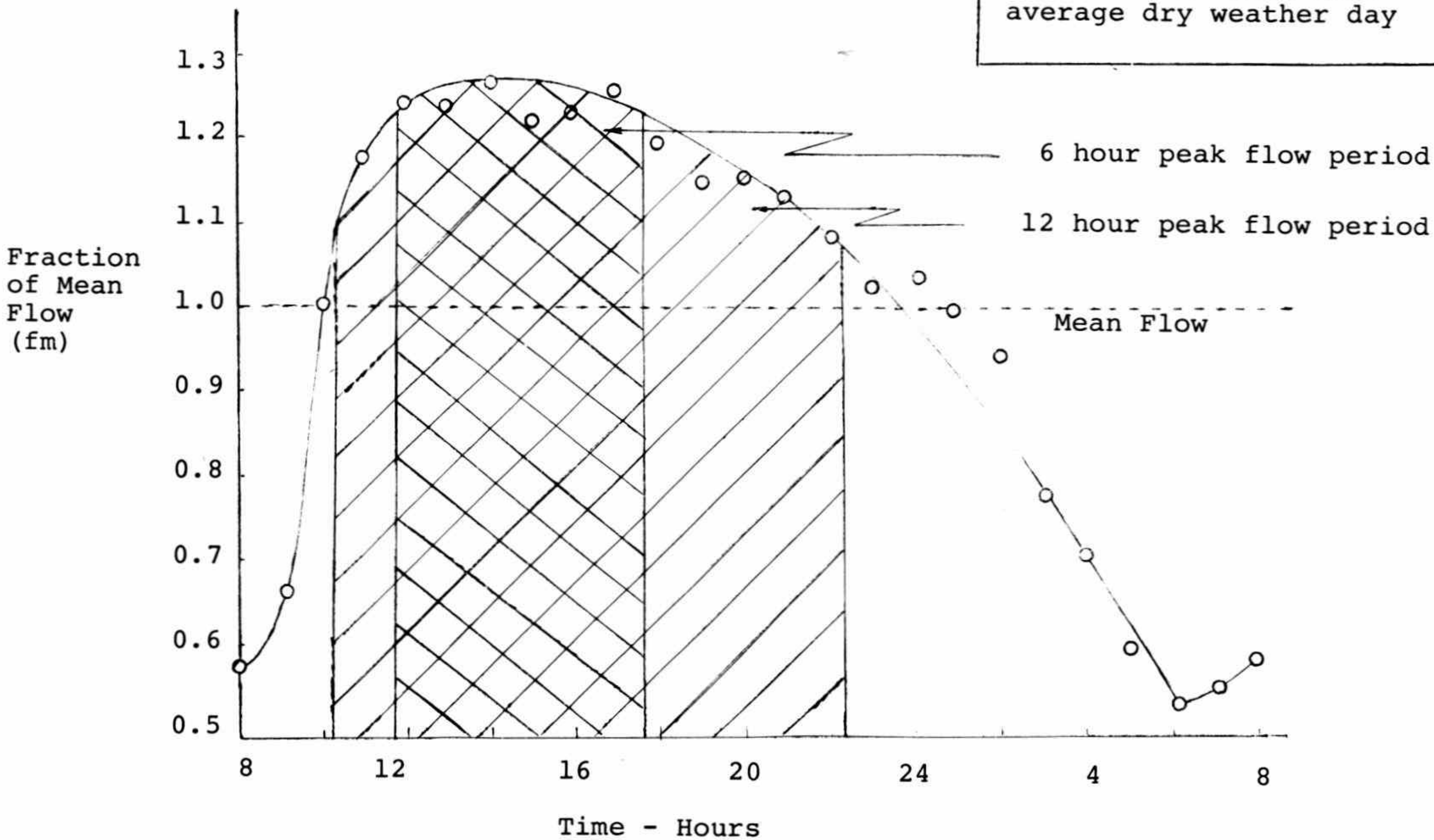
4.1 EQUALIZATION BASIN SIZE

4.1.1 Determination of Flow Variations

The diurnal flow for an average dry weather day is plotted in Figure 4.1. The points on the graph are the average of the data from several days of measurement. Inspection of Figure 4.1 indicates that the ratio of peak flow to mean flow is 1.28 to 1, the ratio of the 6 hour peak flow rate to the mean flow rate is 1.26 and the ratio of the 12 hour peak flow rate to the mean flow rate is 1.22. Information on the diurnal flow variations at Plant "A" during periods of high flow were not available.

FIGURE 4.1

Variation in flow over average dry weather day



The frequency of occurrence of mean daily flows over a one year period are plotted in Figure 4.2. From these data the annual daily mean flow was found to be 40.5 MGD (153,292 cu. m/day) and the peak daily mean flow was 63 MGD (238,455 cu. m/day). Thus the peaking factor for seasonal variation in flows was 1.55.

4.1.2 Storage Volume

The accumulation of hourly flows on the average dry weather day in the form of fraction of mean flow were abstracted from Figure 4.1 and were plotted as shown in Figure 4.3. A straight line plot representing the accumulative flow of uniform discharge from an equalization basin is also shown in the Figure. Hence, the total fraction of flow requiring storage to produce the uniform discharge is represented by the sum of the vertical distances between the separate plots at points "a" and "b"

$$v_m = v_{m_1} + v_{m_2}$$
$$= 1.99 + 0.74 = 2.73 \qquad 4.1$$

The equalization volume required to control any daily flow was determined using equation 3.1 resulting in

$$V_E \text{ (cu ft)} = \frac{v_m}{24} \frac{Qm}{6.24} = \frac{2.73}{24 \times 6.24} Qm = .0182\ Qm \qquad 4.2$$
$$V_{EM} \text{(cu m)} = V_E \text{ (cu ft)} \times .0283$$

The maximum volume required, i.e. to equalize the flow on the peak day for the year under study, equals

$$V_E = .0182 \times 63 \times 10^6 = 1.15 \times 10^6 \text{ cu ft } (3.26 \times 10^4 \text{ cu m})$$

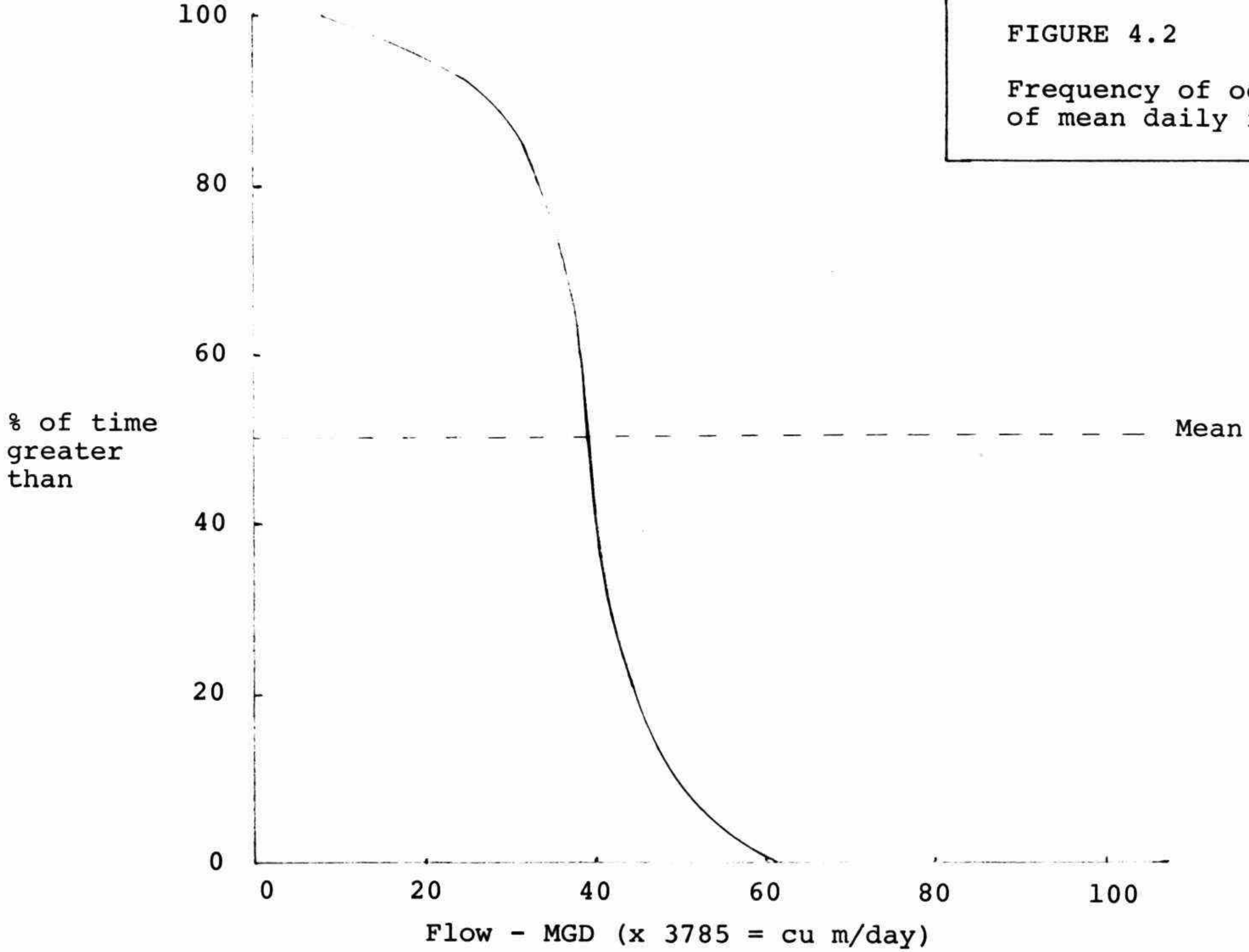
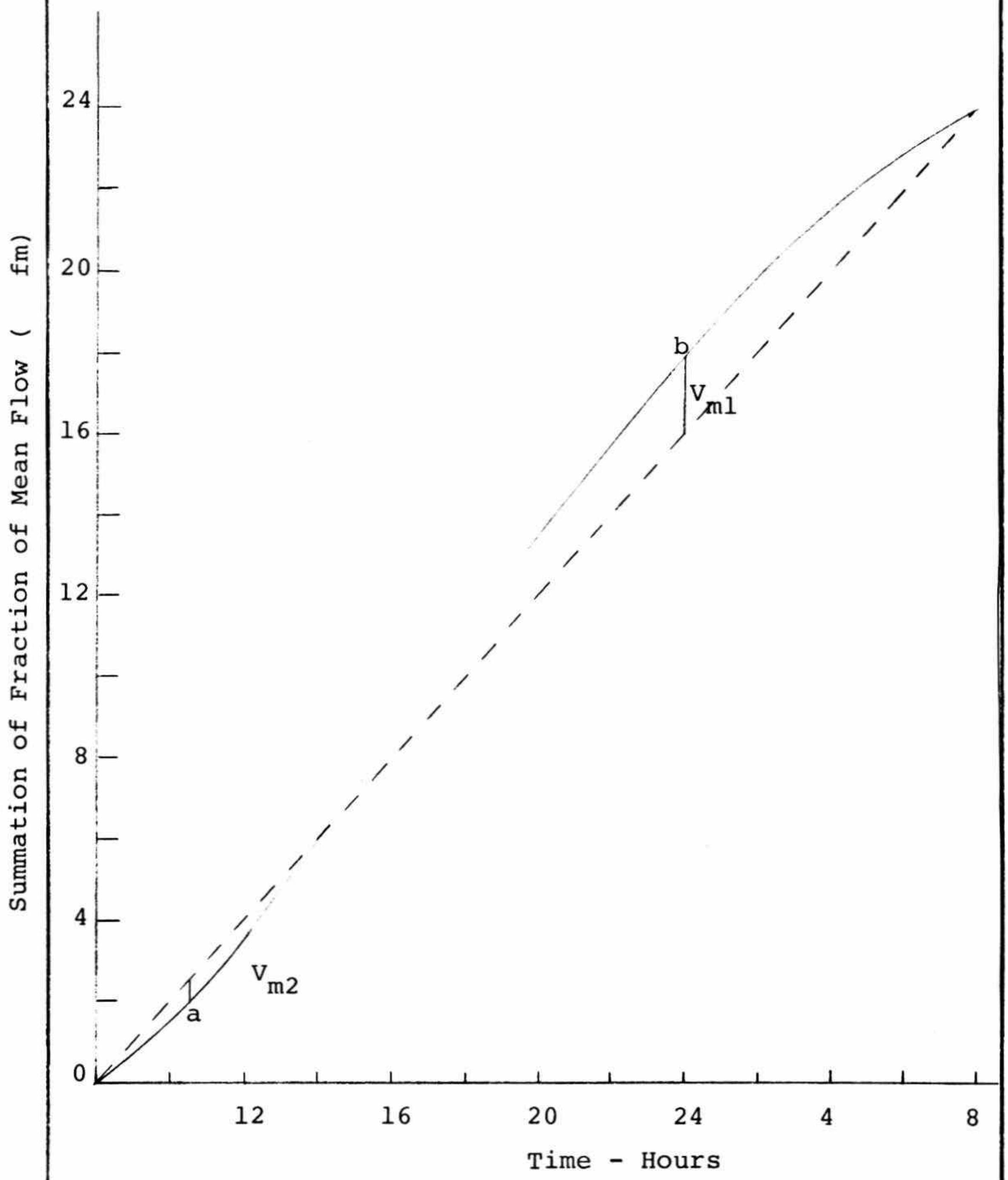


FIGURE 4.2

Frequency of occurrence of mean daily flows

FIGURE 4.3
Cumulative flow for 24-hour Period



4.1.3 Cost of Storage

The percentage of the time a specific storage volume would be required was calculated using Equation 4.2 and the flow frequency curve shown in Figure 4.2. The equalization volume frequency relationship has been plotted on Figure 4.4. The curve indicates that a storage volume greater than zero is required 100 percent of the time and a volume greater than 1.15×10^6 cu. ft. (3.26×10^4 cu. m.) is required 0 percent of the time. The storage volume required 50 percent of the time, i.e. to equalize the annual daily mean flow, was found to be 840,000 cu. ft. (21,000 cu. m.).

In developing the costs of equalization facilities, it was established that for the purposes of this study the costs would be based on concrete tanks constructed below ground level. These costs are indicated in Figure 4.5.

Using cost data presented in Figure 4.5 and the storage volume requirement information presented in Figure 4.4, the relationship between cost and storage volume frequency requirements were obtained and are plotted in Figure 4.6. This curve indicates that to provide equalization to the annual daily peak flow, i.e. a maximum storage volume of 1.15×10^6 cu. ft. (3.26×10^4 cu. m.), would cost approximately $535,000 and to provide equalization facilities for the annual daily mean flow would cost about $300,000.

The methodology developed can be used in determining the extent that a given plant will handle peak flows to eliminate or reduce flow by-passing. Although the cost savings in the following sections are based on total retention of the maximum day flow, the equalization basin design methodology permits the selection of some lesser level of flow control to a plant and the estimation of the frequency of occurrence of periods

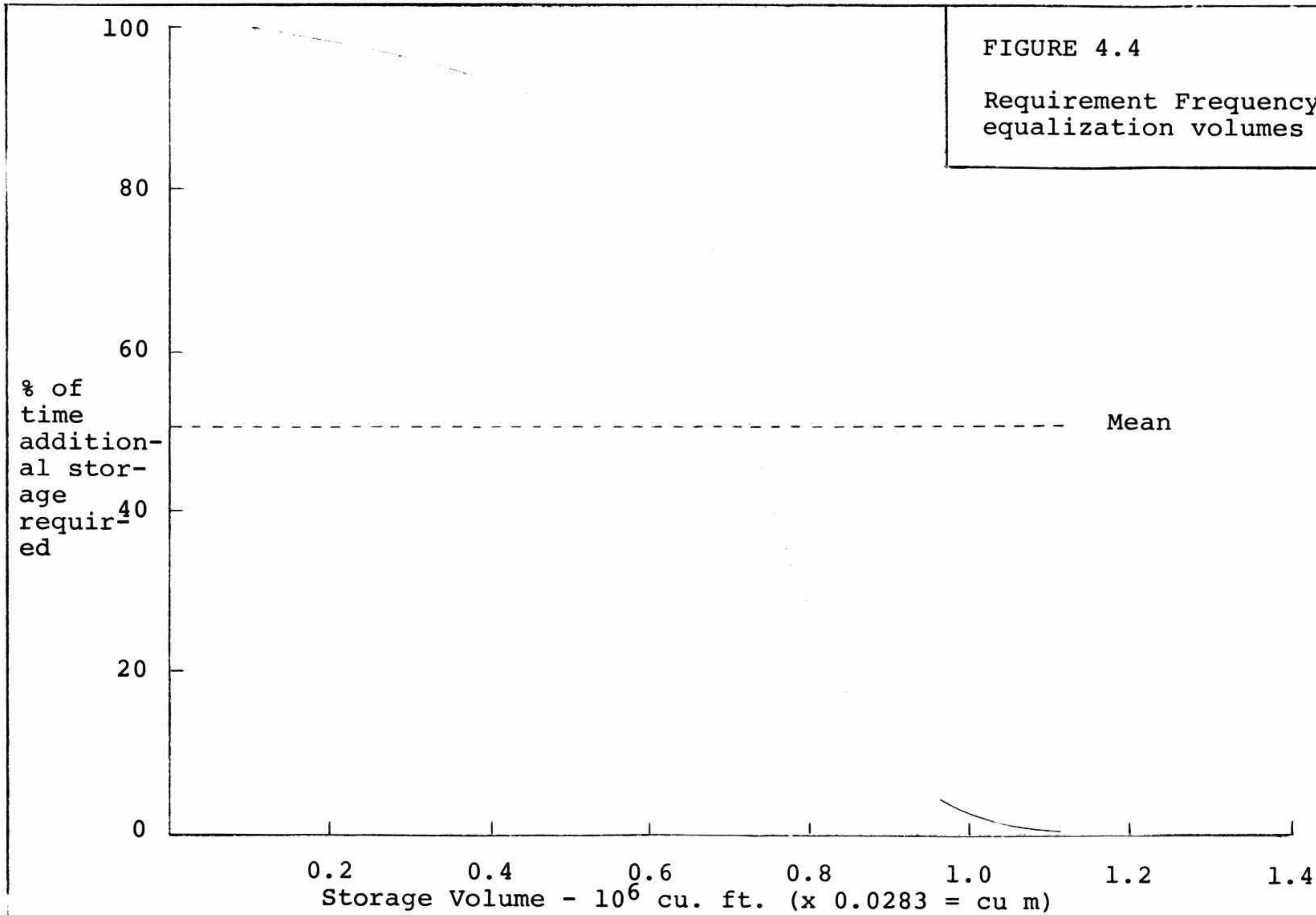


FIGURE 4.4

Requirement Frequency for equalization volumes

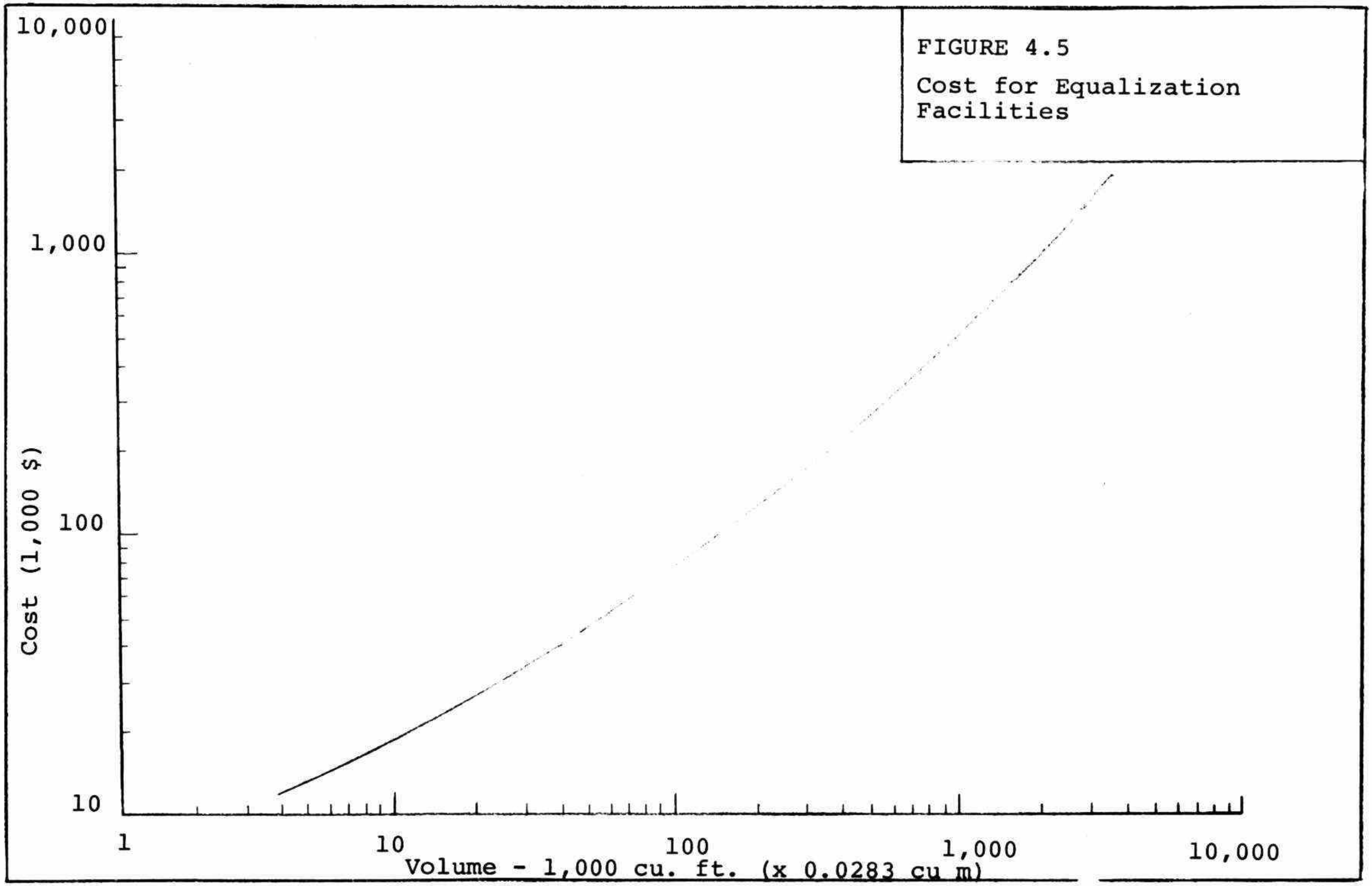


FIGURE 4.5

Cost for Equalization Facilities

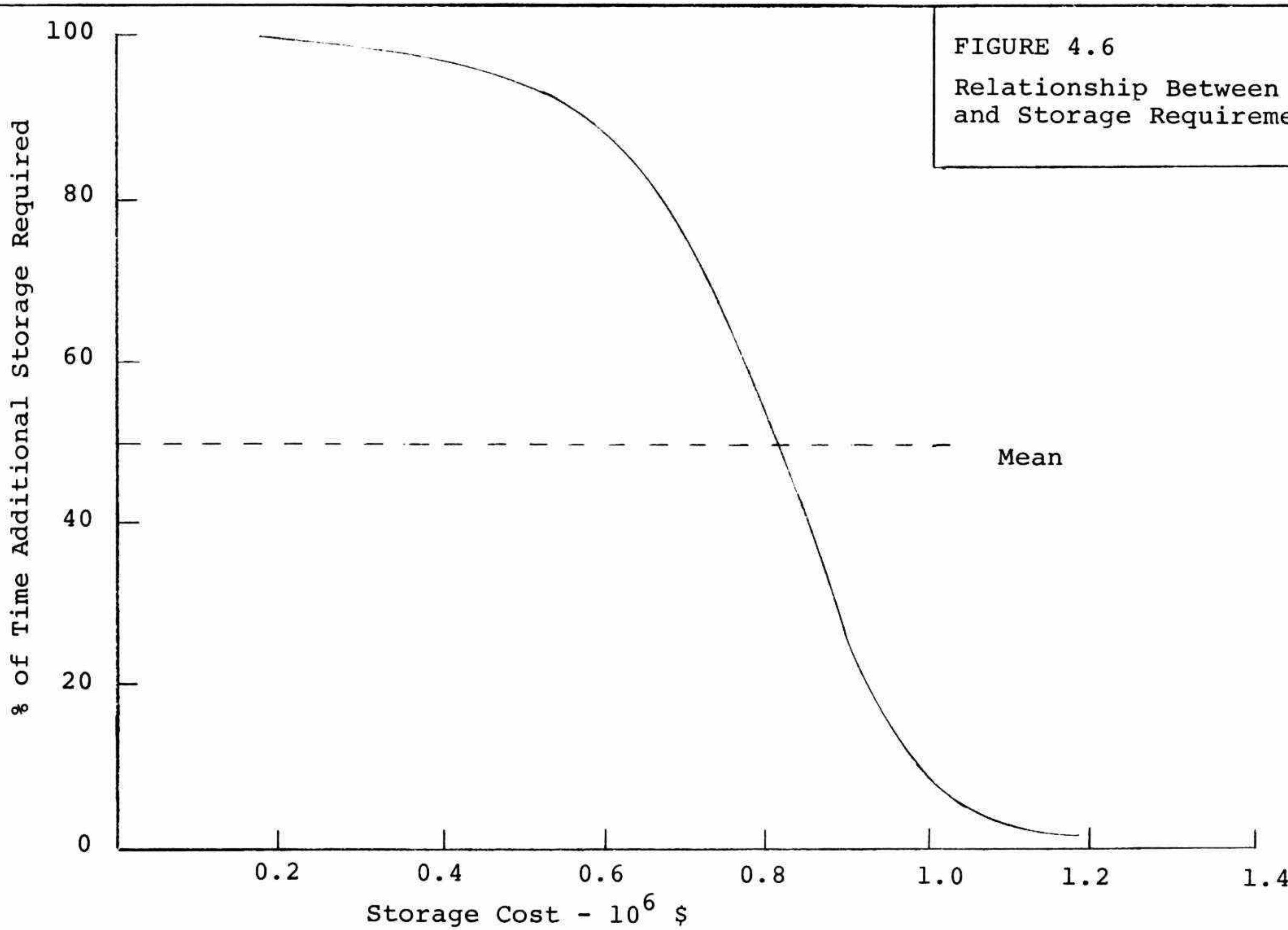


FIGURE 4.6

Relationship Between Cost and Storage Requirements

with non-equalized flow.

4.2 PRIMARY CLARIFIER DESIGN

4.2.1 Cost of Existing Facilities

The methodology described in Section 3 for the determination of the area of primary clarifiers required under fluctuating flow conditions was used in the design of Plant "A". In order to remain consistent, the cost of these facilities was determined from the cost curve shown in Figure 4.7. This cost data was obtained from the EPA report, "Estimating Costs and Manpower Requirements for Conventional Waste Water Treatment Facilities".(5) The area and cost of the primary clarifiers for Plant "A" are:

Total Area - 37,900 sq. ft. (3,716 sq. m.)
Total Cost - $630,000

2 clarifiers - 10,400 sq. ft. (966 sq. m.)
- $185,000 each
1 clarifier - 16,500 sq. ft. (153 sq. m.)
- $250,000

The operating characteristics of Plant "A" can be presented in graphical form as described in Section 3. Figure 4.8 presents the relationship between suspended solids removal and overflow rate in the primary clarifiers for one year's operation. The suspended solids removal efficiency was obtained by dividing the difference in the solids load between the influent and effluent streams by the influent solids load. The influent solids values include the raw sewage solids and the solids in the waste activated sludge because in this plant the waste activated sludge is mixed with the raw sewage in the preaeration facilities.

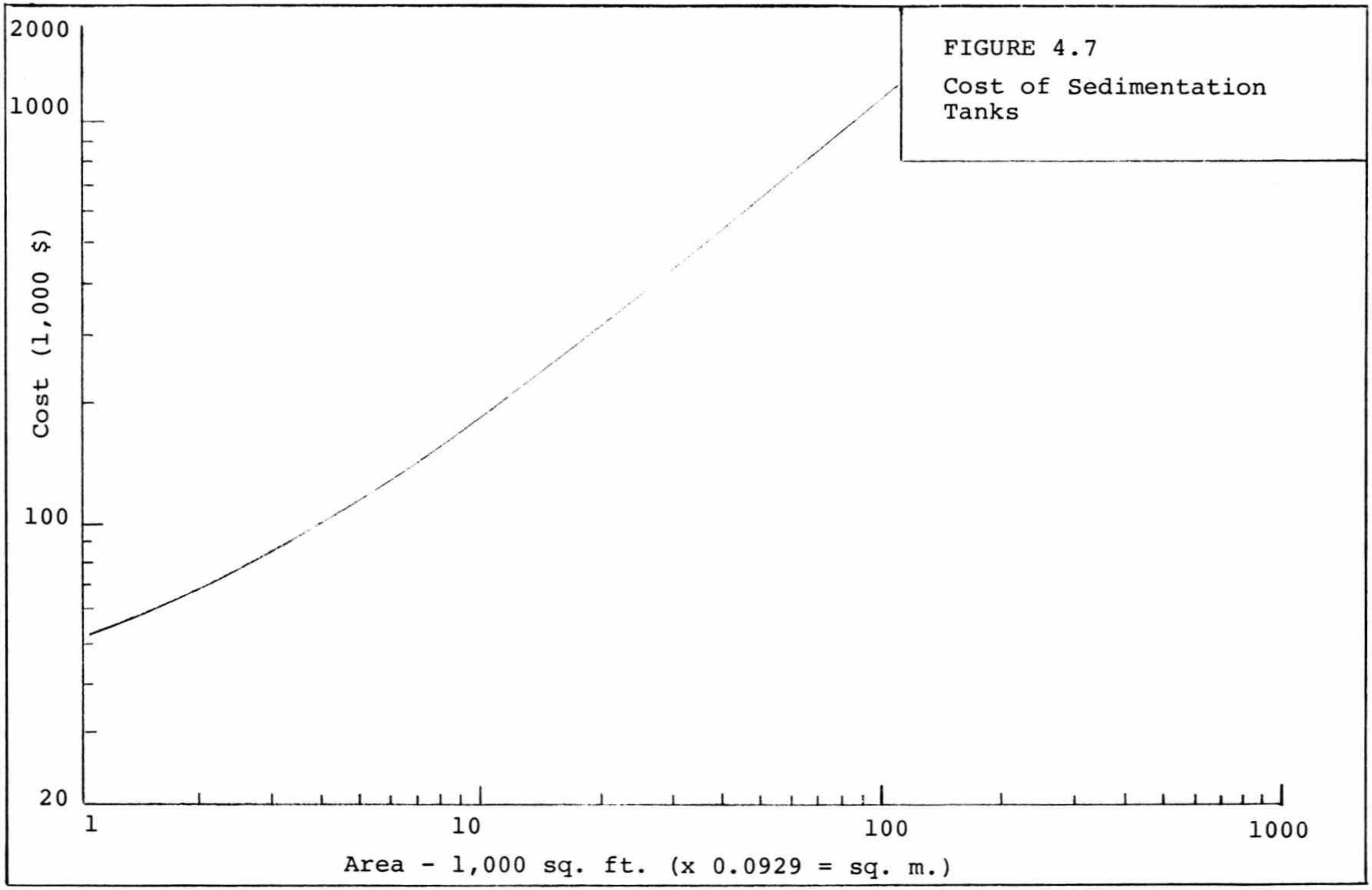


FIGURE 4.7
Cost of Sedimentation Tanks

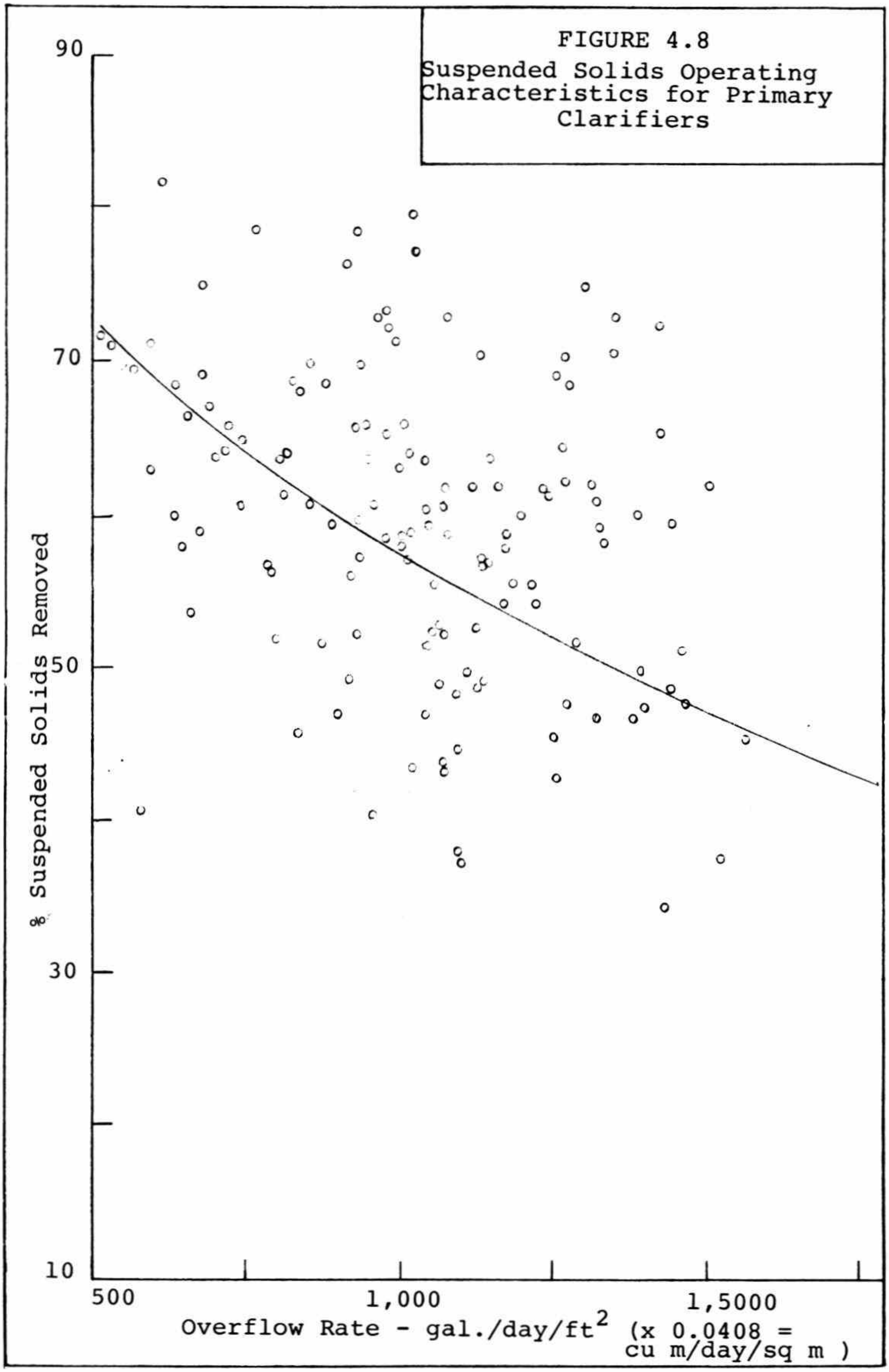


FIGURE 4.8
Suspended Solids Operating Characteristics for Primary Clarifiers

The results in Figure 4.8 are seen to vary widely and show no consistent trend. Reasons for the lack of data correlation may be intermittent activated sludge wasting, difficulties in obtaining representative samples in a full-scale plant and other factors which at this time are unknown, but which may relate to short-term flow fluctuations. However, so that the theoretical developments of Section 3 could be applied a curve based on a typical curve from the ASCE and WPCF Sewage Treatment Design Manual(1) was plotted in the same figure and used to estimate suspended solids removal efficiencies at different flow rates.

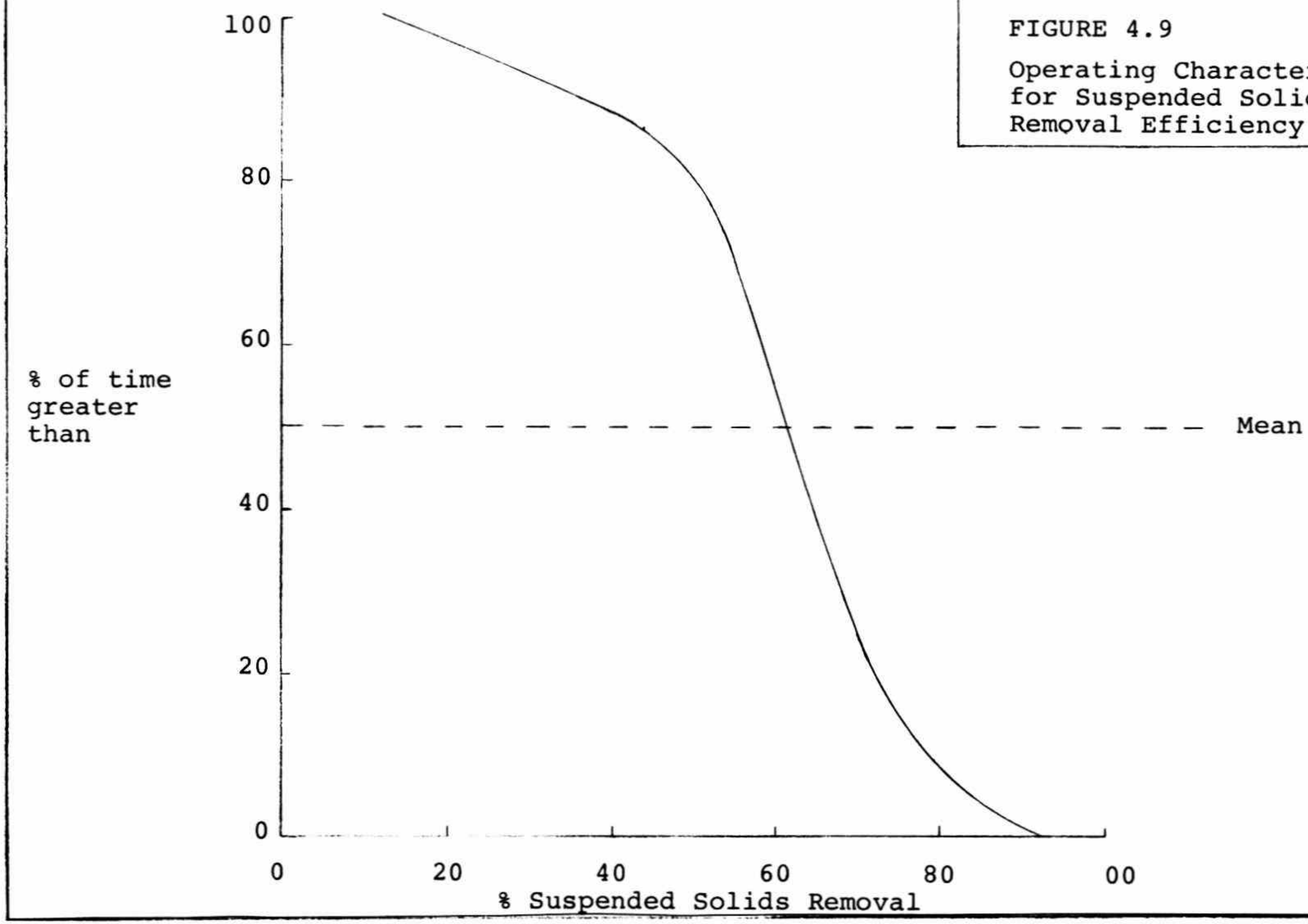
The frequency of occurrence of the suspended solids removal efficiencies has been plotted in Figure 4.9 using the Plant "A" data. The graph shows that the mean suspended solids removal efficiency for the year under consideration was 62.5 percent.

The relationship between BOD removal and overflow rate in the primary clarifier could not be calculated for Plant "A" as the BOD of the waste activated sludge stream, which is fed into the primary tanks, is unknown.

The design of primary clarifiers operating under equalized conditions can be compared to the primary clarifiers in Plant "A" on the basis of equivalent annual mean suspended solids removal efficiencies as described in Section 3. In actual operation it would be expected that equalized flows would produce higher efficiencies of suspended solids removal than those indicated in the following calculations but the data available are not sufficiently complete to permit an estimate of the expected improvement.

FIGURE 4.9

Operating Characteristics for Suspended Solids Removal Efficiency



4.2.2 Suspended Solids Removal Versus Overflow Rate Relationship for Equalized Flow Conditions

In Section 3 it was indicated that the suspended solids removal efficiency under equalized flow conditions could be obtained from fluctuating flow data using either sub-day data or the factor, f_e, which transforms daily data for varying flow conditions to equalized flow conditions. Sub-day data on the suspended solids removal efficiency of the primary clarifiers were not available from Plant "A". Thus it was necessary to apply the factor f_e from Equation 3.5 to the solids removal curve shown in Figure 4.8 for varying flow conditions. The relationship for suspended solids removal at various overflow rates under equalized conditions is presented in Figure 4.10.

4.2.3 Calculation of Clarifier Size with Equalized Flows

To determine the size of primary clarifiers to be used under equalized flow conditions a trial and error technique was used. The yearly flow pattern can be broken down into constant flow periods that would be controlled by means of the equalization basin. Figure 4.11 describes the actual variations in flow in Plant "A" over one year's operation. Superimposed on this plot are one week average and two week average constant flow periods which represent the constant outflow from an equalization basin.

The clarifier requirements were determined by selecting an area A_n, and for each constant flow period calculating the associated overflow rate. Using Figure 4.10 a suspended solids removal efficiency was calculated for each overflow rate. From this information the frequency of occurrence of particular suspended solids removal efficiencies were calculated and plotted in Figures 4.12 and 4.13. The annual daily mean values of these plots were compared to the daily

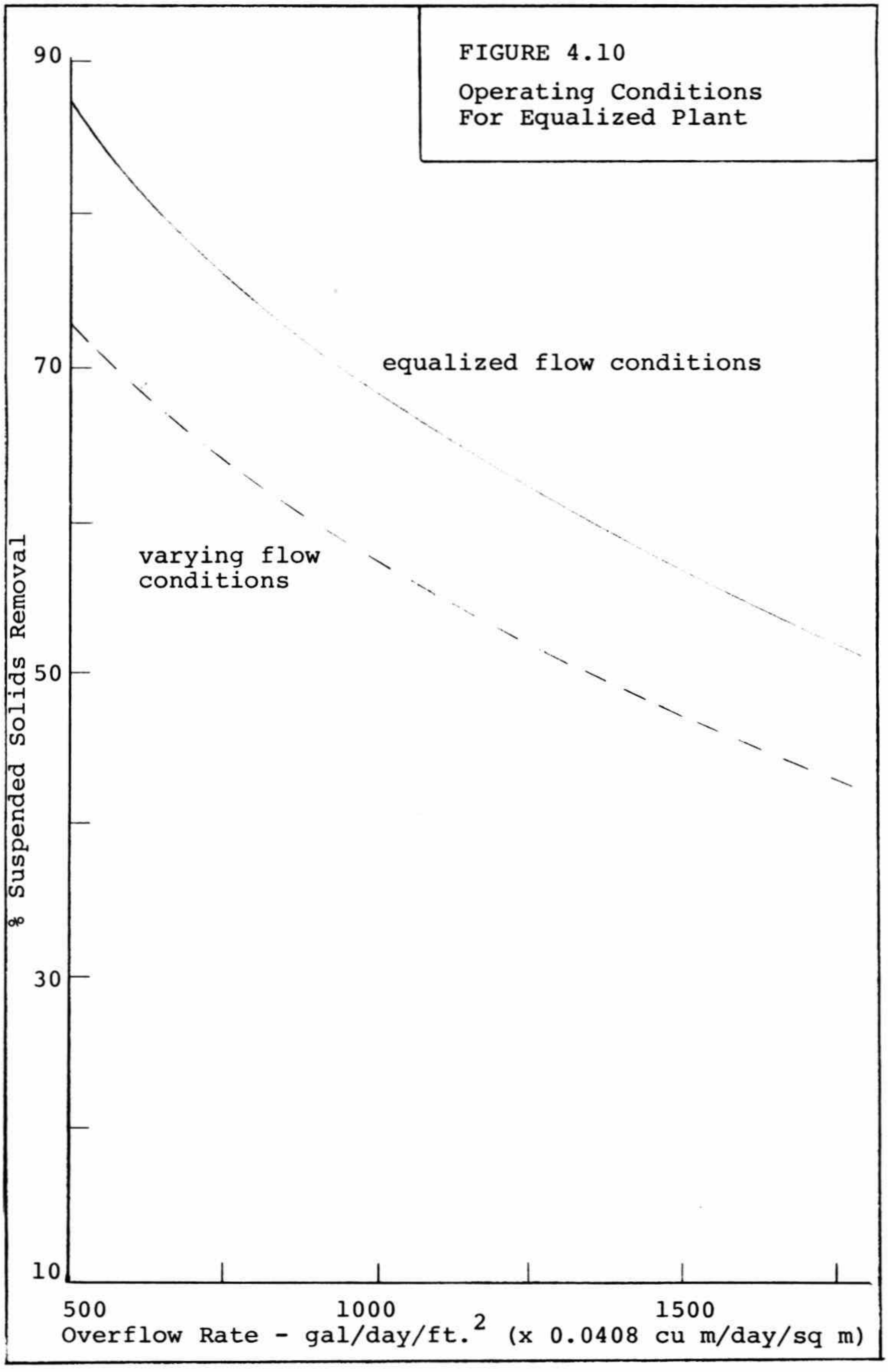


FIGURE 4.10
Operating Conditions
For Equalized Plant
equalized flow conditions
varying flow
conditions
% Suspended Solids Removal
90
70
50
30
10
500
1000
1500
Overflow Rate - gal/day/ft.2 (x 0.0408 cu m/day/sq m)

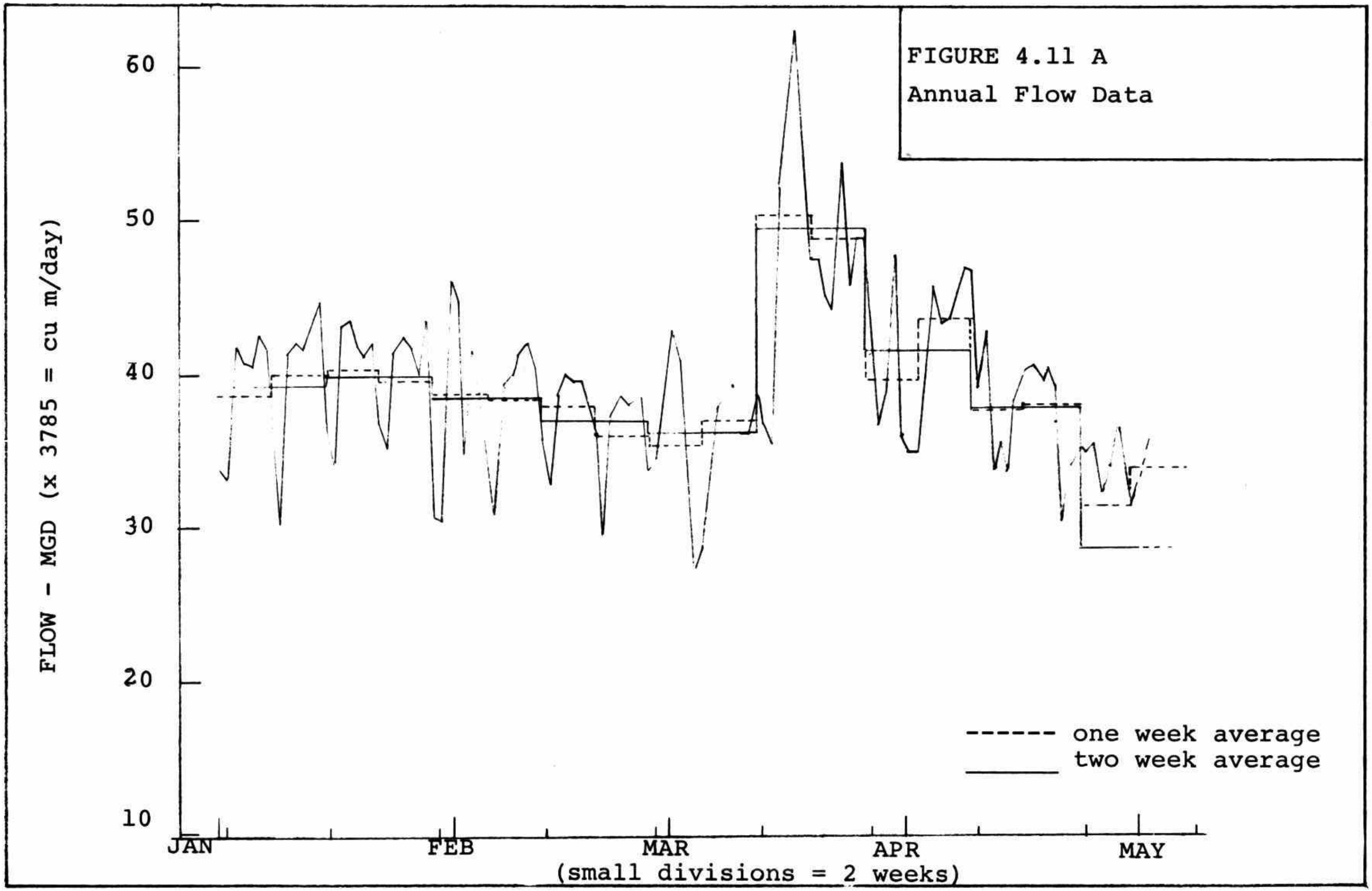


FIGURE 4.11 A
Annual Flow Data
FLOW - MGD (x 3785 = cu m/day)
60
50
40
30
20
10
JAN
FEB
MAR
APR
MAY
(small divisions = 2 weeks)
------ one week average
two week average

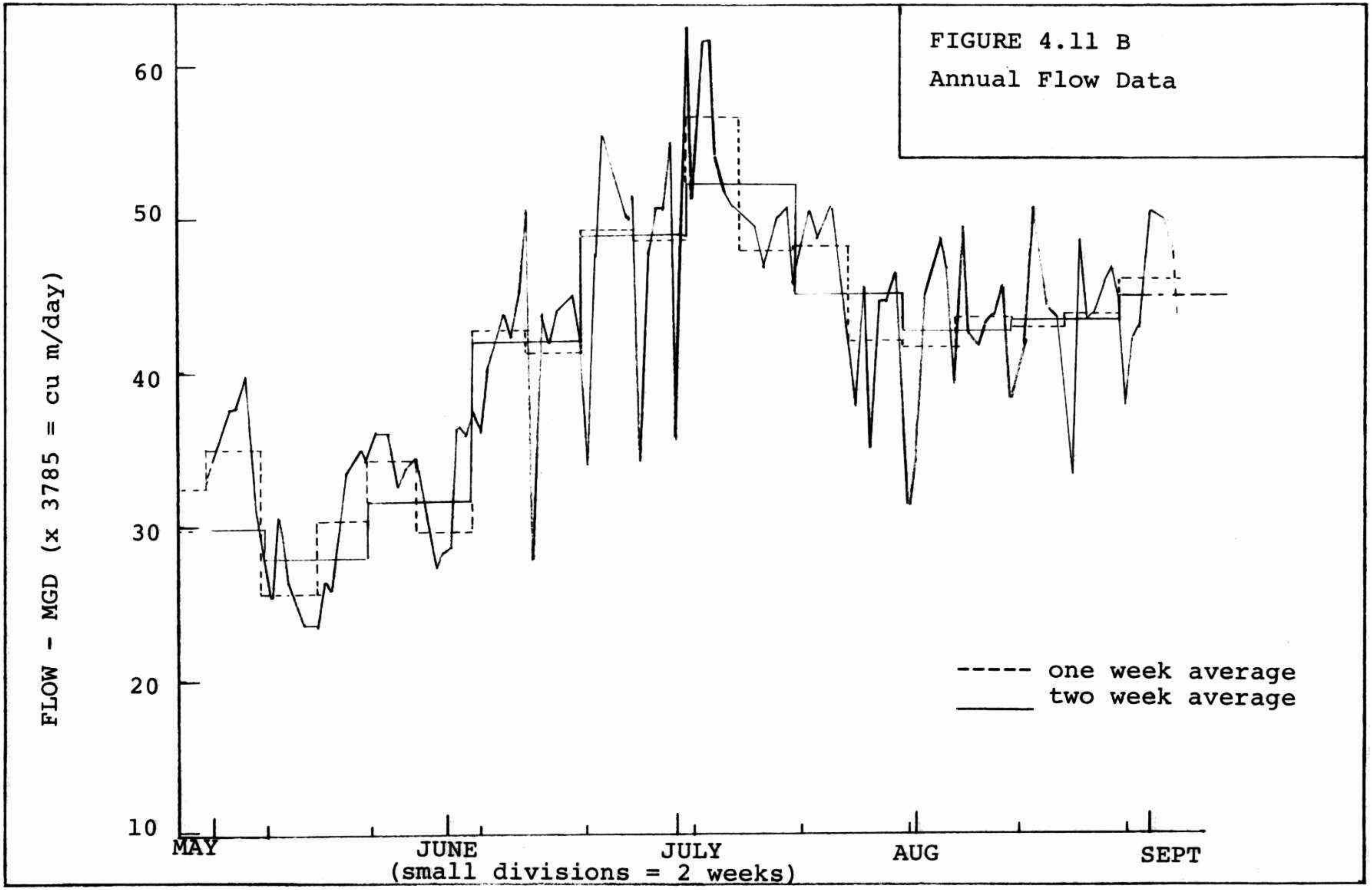


FIGURE 4.11 B
Annual Flow Data

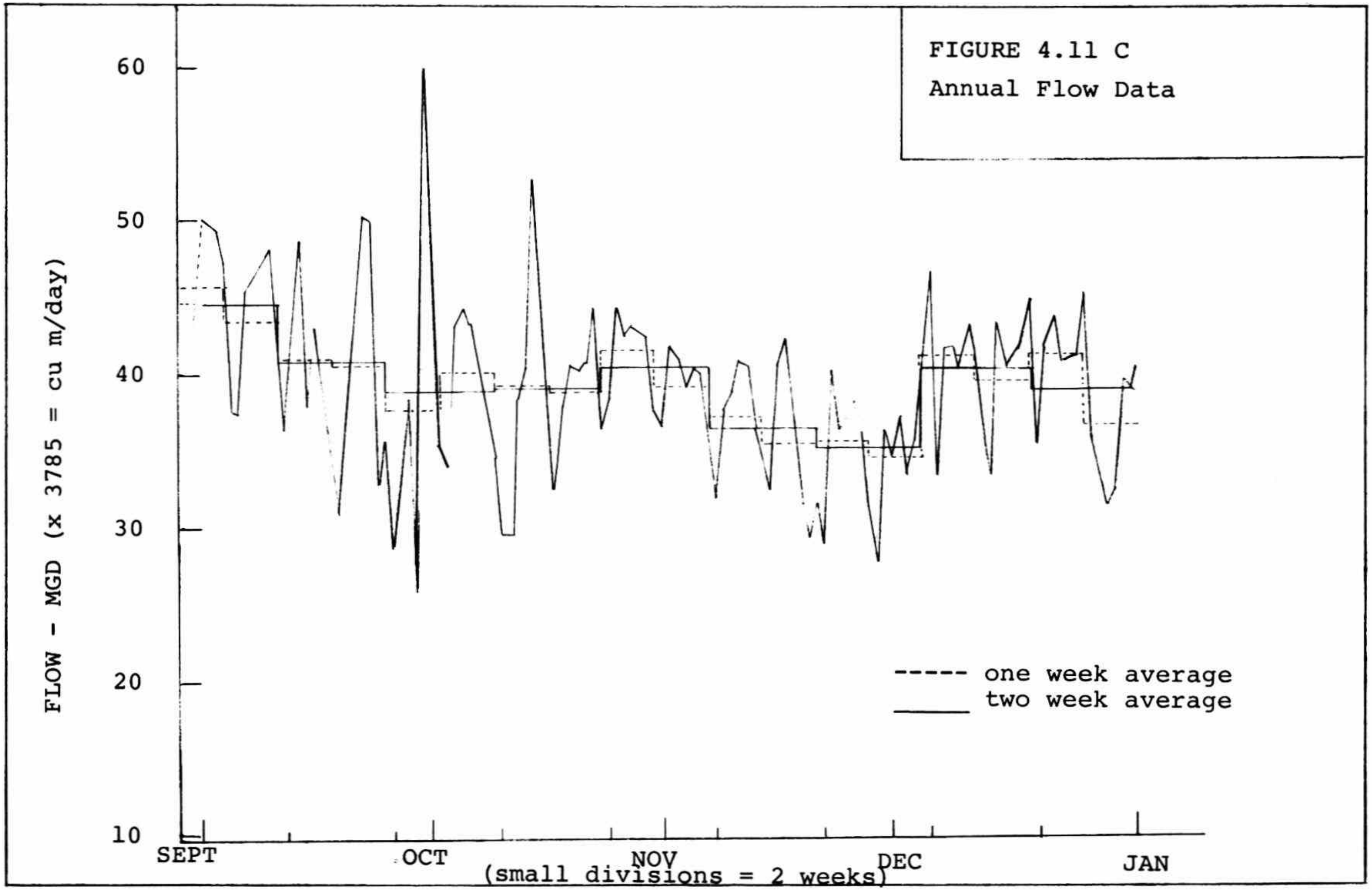


FIGURE 4.11 C
Annual Flow Data

mean value for fluctuating flow of 62.5 percent.

If agreement was not obtained a new area was selected. The plots resulting from the trials using the two week average are presented in Figure 4.12 and the one week average in Figure 4.13. A summary of the mean values for the various trials is given in Table 4.1. The primary clarifier area was found to be 32,400 sq. ft. (3,010 sq. m.). As agreement was obtained between the one week average and two week average calculations, a calculation based on daily flows was not carried out.

Assuming that the two smaller clarifiers at the fluctuating flow plant would remain the same size and that the larger clarifier would be reduced in size, the total cost for primary clarifiers for equalized flow conditions was estimated from Figure 4.7 to be;

Total Area (equal.)	32,400 sq. ft. (3,010 sq. m.)
Total Cost (equal.)	$565,000

2 clarifiers	- 10,400 sq ft (966 sq m)
	$185,000 each
1 clarifier	- 11,600 sq ft (1,078 sq m)
	$195,000

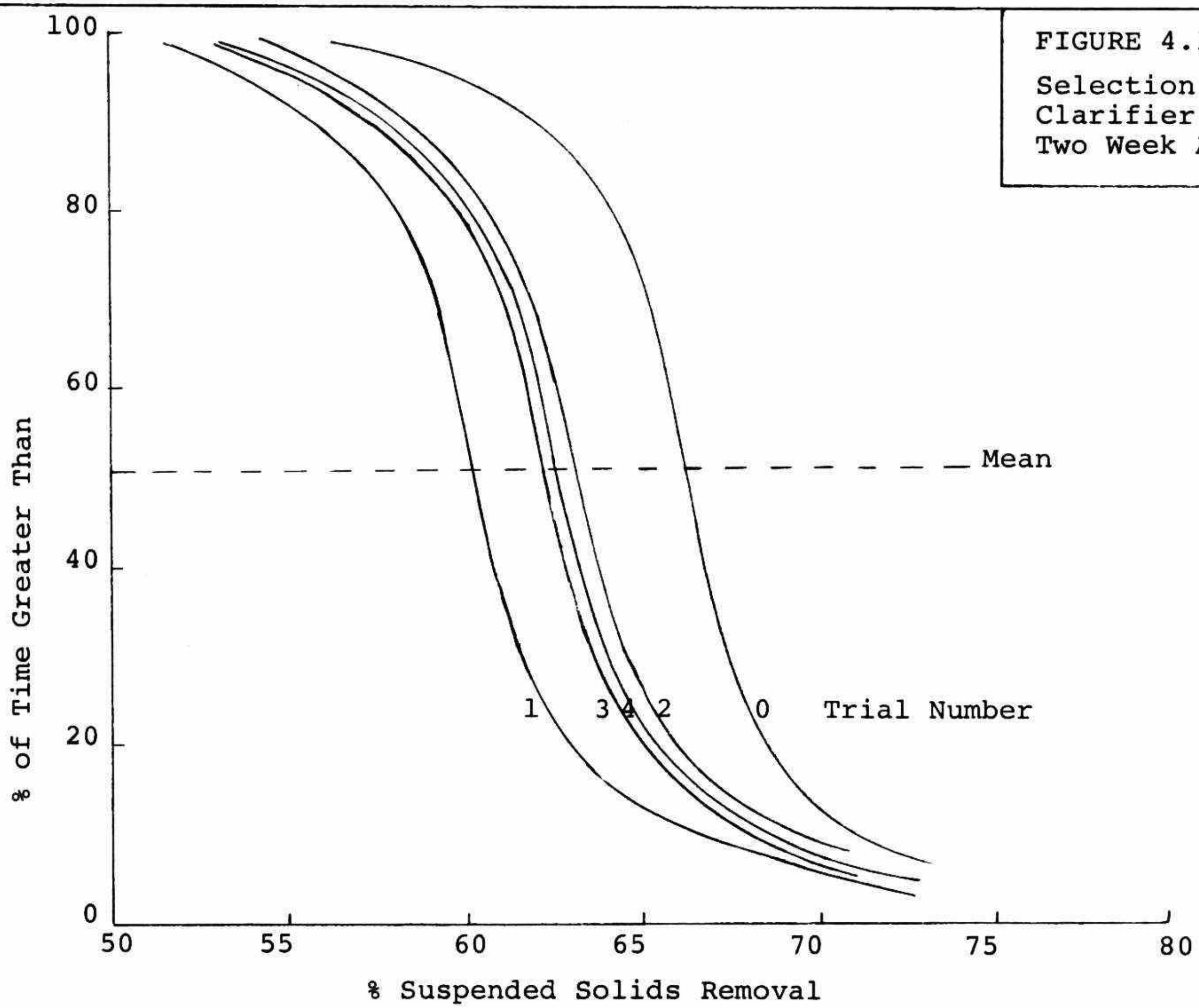
Thus the cost saving in primary clarification by using an equalization basin to control the maximum 24 hour flow would be $65,000.

4.2.4 Comparison of Equalized Flow Versus Fluctuating Flow on Basis of Improved Efficiency

The addition of an equalization basin to the existing facilities at Plant "A" would improve the operating efficiency of the plant. Based on available operating information,

FIGURE 4.12

Selection of Primary Clarifier Area
Two Week Average Flow



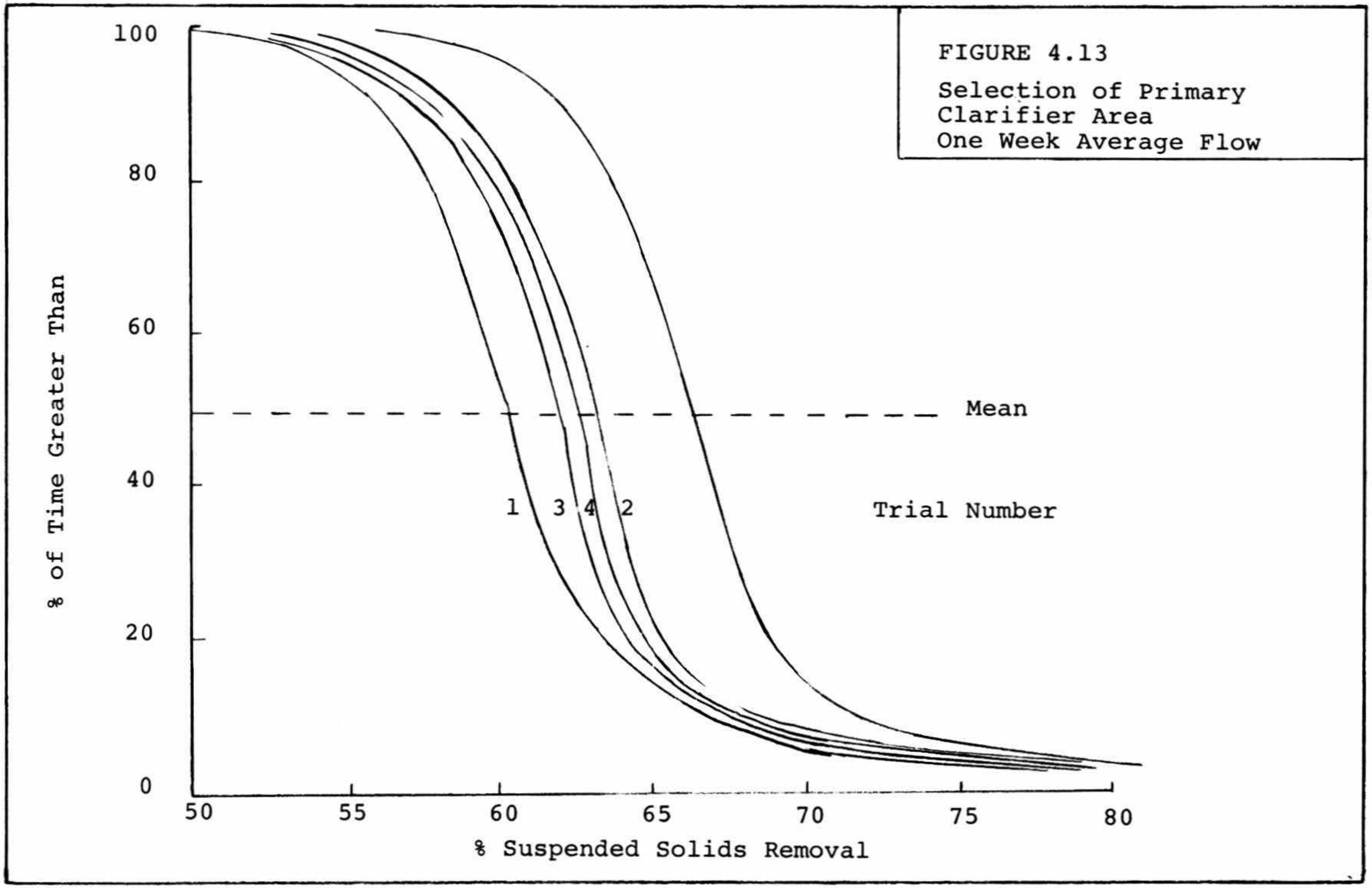


FIGURE 4.13
Selection of Primary Clarifier Area
One Week Average Flow

TABLE 4.1

DETERMINATION OF PRIMARY CLARIFIER SIZE

A. Two Week Average Flows

Trial Number	0	1	2	3	4
Area (ft.^{2}) (sq m)	37,000 (3,716)	30,000 (2,787)	33,000 (3,066)	32,000 (2,973)	32,400 (3,010)
Mean Overflow Rate (gal/day/ft^2) (cu m/day/sq m)	1,060 (52)	1,329 (65)	1,208 (59)	1,246 (61)	1,230 (60)
Suspended Solids Removal Efficiency (%)	66.4	60.2	63.0	62.1	62.5

B. One Week Average Flows

Trial Number	0	1	2	3	4
Area (ft.^{2}) (sq m)	37,300 (3,716)	30,000 (2,787)	33,000 (3,066)	32,000 (2,973)	32,400 (3,010)
Mean Overflow Rate (gal/day/ft^2) (cu m/day/sq m)	1,067 (52)	1,327 (65)	1,206 (59)	1,244 (61)	1,230 (60)
Suspended Solids Removal Efficiency (%)	66.5	60.3	63.1	62.2	62.5

the improved suspended solids removal efficiency for 100 percent equalization of the maximum day flow is shown in Figures 4.12 and 4.13 and Table 4.1 as Trial Number 0 which has the same clarifier surface area as the existing Plant. The suspended solids removal efficiency is seen to be 66.5 percent for varying flow conditions in the present plant.

4.3 AERATION PLANT DESIGN

As was indicated in Section 3, selection of aeration tank capacities for equalized and fluctuating flow conditions should be based on BOD removal rate information. An illustration of the biological capabilities of Plant "A", operating under varying flow conditions, is given in Figure 4.14 and 4.15. The mean BOD removal by secondary treatment is shown in Figure 4.14 to be 85 percent. The variation over the year of the first order BOD removal rate constant is presented in Figure 4.15. The k value for Plant "A" in the year this data was taken was found to have a mean value of 0.00602 $days^{-1}$ with a standard deviation of 0.00381 $days^{-1}$

A review of the literature has indicated that there is insufficient information relating BOD removal with kinetics for full-scale plants operating under equalized and varying flow conditions. For the purposes of this study it was assumed that the aeration tank capacity required for the two operating conditions would be equal although it is anticipated that less aeration tank capacity would be required for the equalized flow case. A comparison in costs between equalized and varying flow biological plants consequently was therefore carried out only on the basis of air requirements and return sludge facilities. As a means of comparison, the costs of these facilities were evaluated based on available operating data.

FIGURE 4.14

BOD removal efficiency of Secondary Treatment

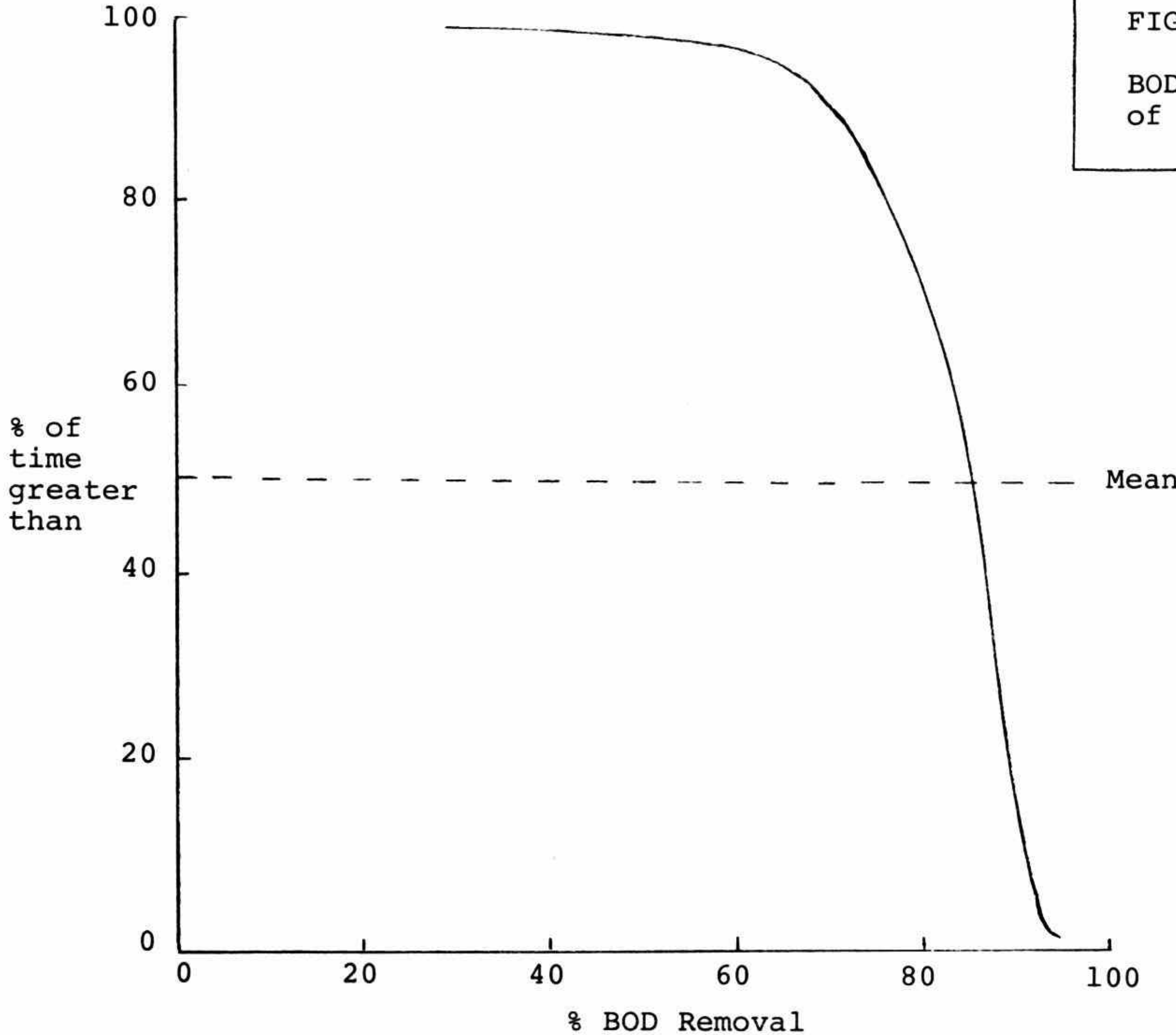
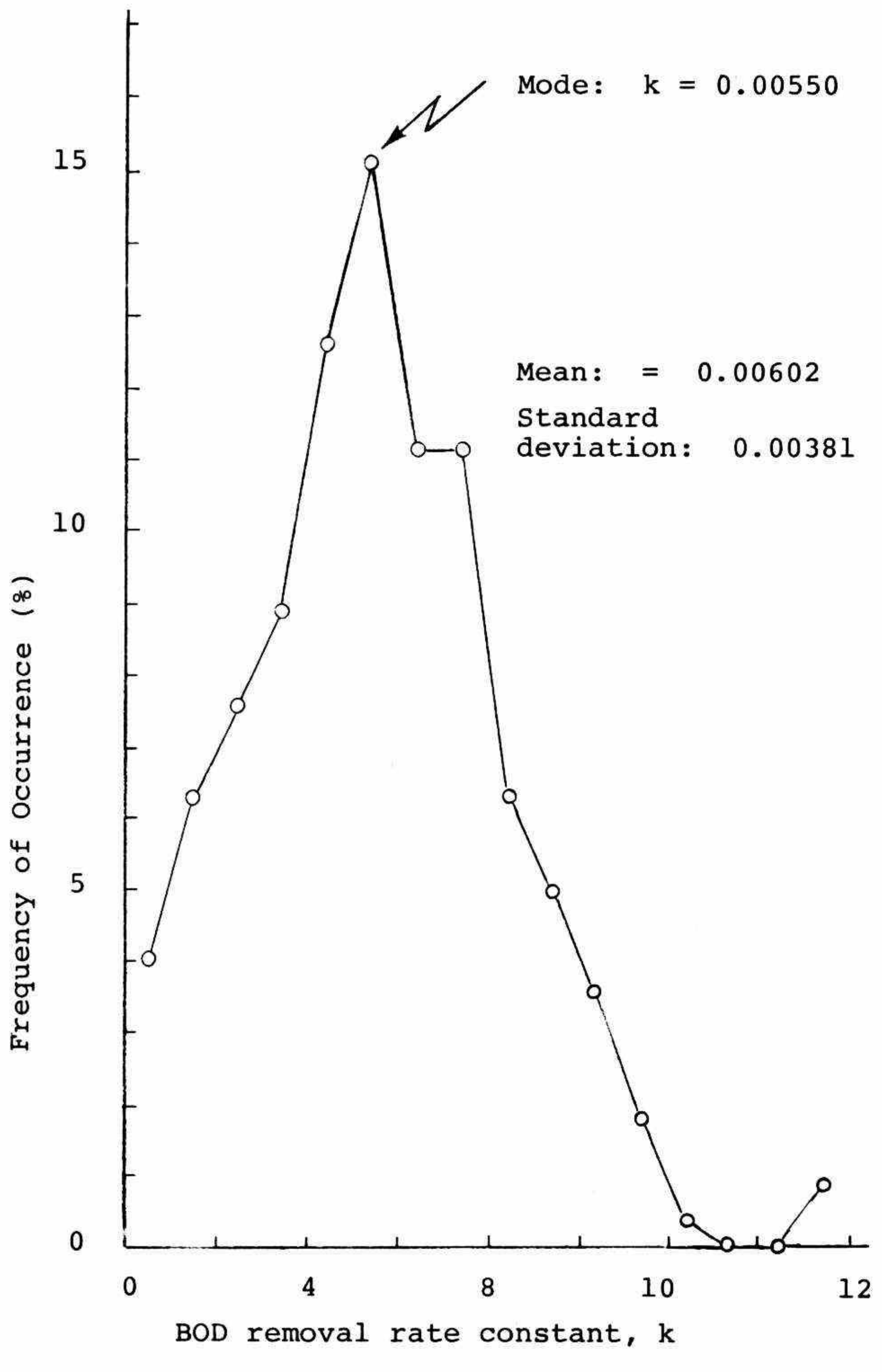


FIGURE 4.15

Biological Activity of the Activated Sludge Plant



4.3.1 Aeration Equipment

The size of air equipment for varying flow conditions can be calculated from Equation 3.13.

$$Sa = 6.95 \; f_a \; Qmax \; BOD_R$$
$$Sam = Sa \times .0283$$

The value of f_a, the maximum 12 hour flow peaking factor on the mean day defined previously in Equation 3.12 is found from Figure 4.1 to be 1.22. The maximum mean daily flow rate is 63 MGD (238,455 cu m/day) as indicated in Figure 4.2. The average BOD level in the primary effluent is 220 mg/l. Assuming a BOD concentration of 15 mg/l in the final effluent the value of BOD_R is 205 mg/l.

The air requirement can thus be calculated to be

$$Sa = 6.95 \times 1.22 \times 63 \times 205 = 110{,}000 \text{ CFM of air}$$
$$Sam = 3{,}113 \text{ cu m/min.}$$

The costs of air supply equipment were obtained from Figure 4.16 (5) which has been updated to current Canadian Conditions. From this figure the cost of supplying 110,000 CFM (3,113 cu m/min) is

COST = \$2,450,000

In a plant having equalized flow conditions the factor f_a becomes equal to 1 resulting in air requirements of

$$\begin{aligned} Sa(equal) &= 6.95 \times 63 \times 205 \\ &= 90{,}000 \text{ CFM of air} \end{aligned}$$

$$Sam\ (equal) = 2{,}547 \text{ cu m/min}$$

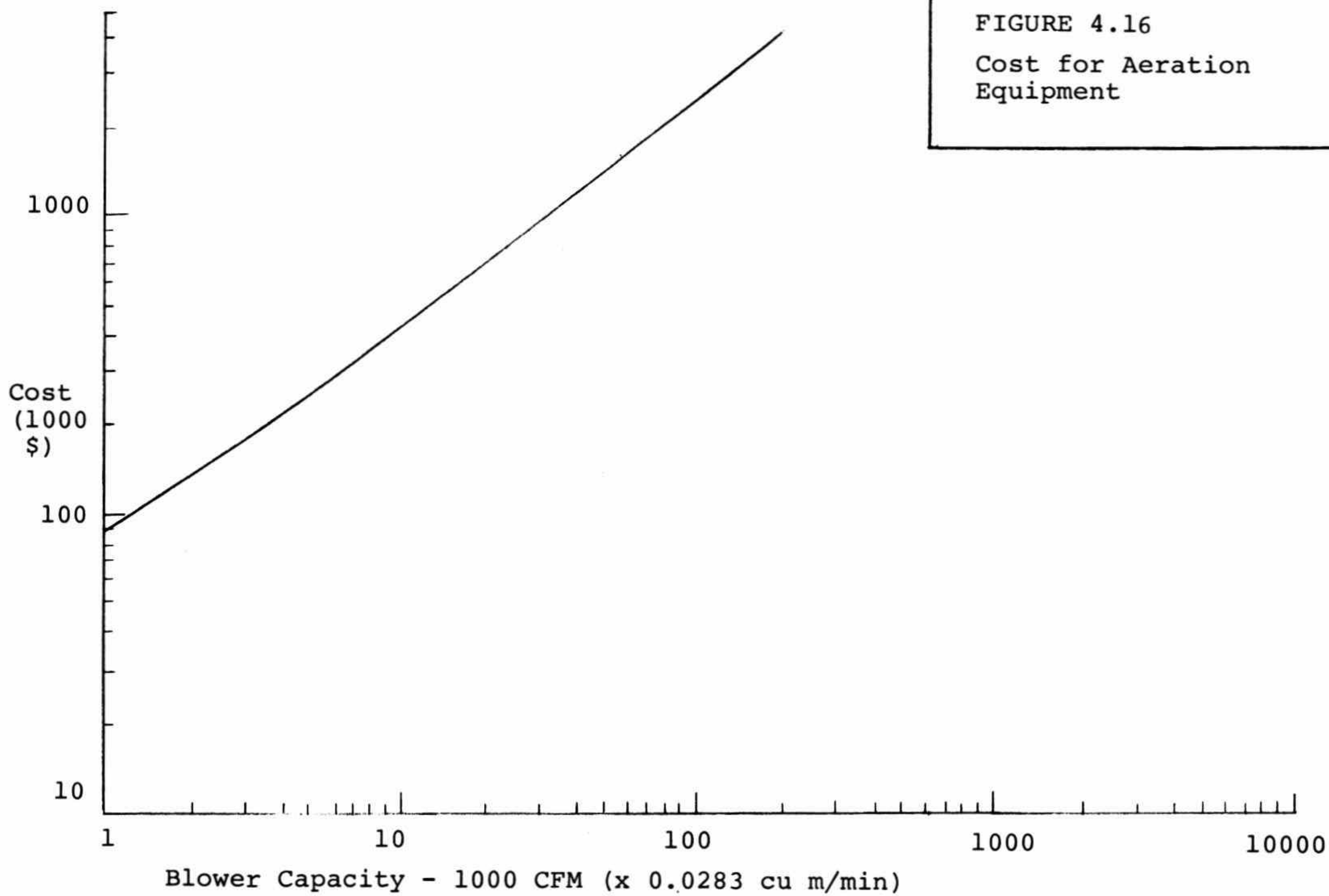


FIGURE 4.16
Cost for Aeration Equipment

and a cost of

COST (equal) = \$2,000,000

Thus the capital cost saving of utilizing equalized flow. conditions would be \$450,000.

Although the peaking in BOD concentrations were not included in the calculations, the results from a series of hourly measurements of the BOD concentration leaving the primary clarifier during dry weather flow conditions from Plant "A" are included in Figure 4.17. The variation of BOD over a day as a function of the mean BOD on that day is plotted with each point on the curve representing the average of several days' data. Comparing Figure 4.17 with Figure 4.1 indicates that the peak in BOD is out of phase with the hydraulic peak. Under equalized flow conditions the BOD curve would be damped and some quality equalization achieved.

4.3.1 Sludge Return Equipment

The maximum sludge recycle rate for Plant "A" under fluctuating flow conditions was calculated from Equation 3.14.

$$R_{max} = f_c \, Qmax \frac{MLVSS}{C_s - MLVSS} \qquad 3.14$$

The following parameters were selected:

$MLVSS$ = 1,700 mg/l
CS = 8,000 mg/l
Q_{max} = 63 MGD (238,455 cu m/day) - Figure 4.2
f_c = 1.26 - Figure 4.1

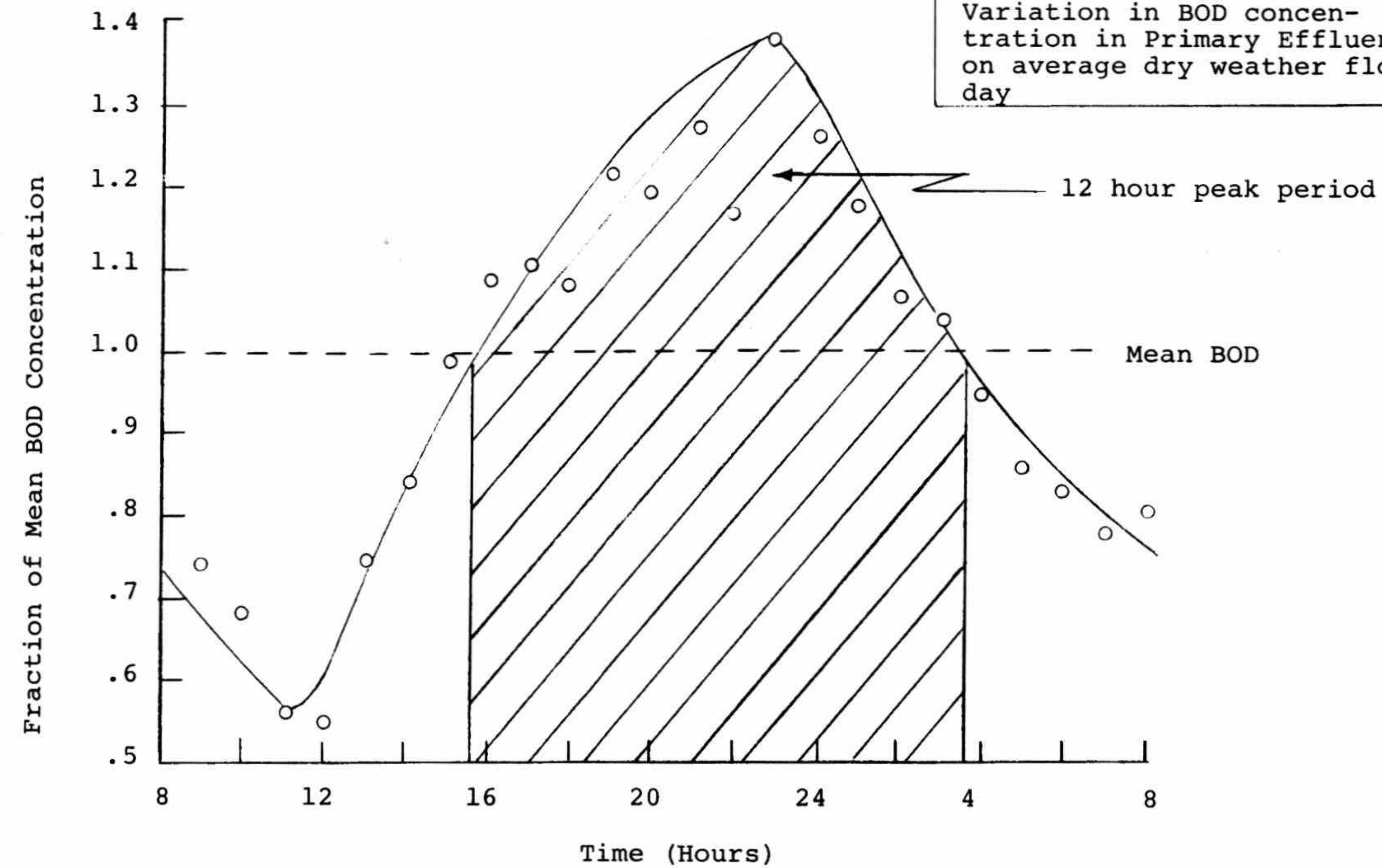


FIGURE 4.17

Variation in BOD concentration in Primary Effluent on average dry weather flow day

This results in

$$R_{max} = 1.26 \times 63 \times \frac{1,700}{8,000 - 1,700} = 21.4 \text{ MGD}$$

(81,139 cu m/day)

The cost for sludge return pumping at various flow rates was obtained from the curve plotted in Figure 4.18.(5) The cost of pumping equipment with a capacity of 21.4 MGD (81,139 cu m/day) equals

Cost = \$275,000

Under equalized flow conditions $f_c = 1$. Using equation 3.14 the required return sludge capacity equals

$$R_{max} \text{ (equal)} = 63 \times \frac{1,700}{8,000-1,700} = 17 \text{ MGD}$$

(64,345 cu m/day)

and the cost from Figure 4.18 is

Cost (equal) = \$112,500

Thus the cost saving realized for equalized flow conditions is \$162,500.

4.4 FINAL CLARIFIER DESIGN

The concepts used to design final clarifiers have been outlined in Section 3. Operating characteristics of the final clarifiers in Plant "A" are presented in Figures 4.19 and 4.20. The suspended solids concentration in the final effluent at various overflow rates is shown in Figure 4.19. The BOD level in the final effluent for various overflow rates is shown in Figure 4.20. On both graphs a line was

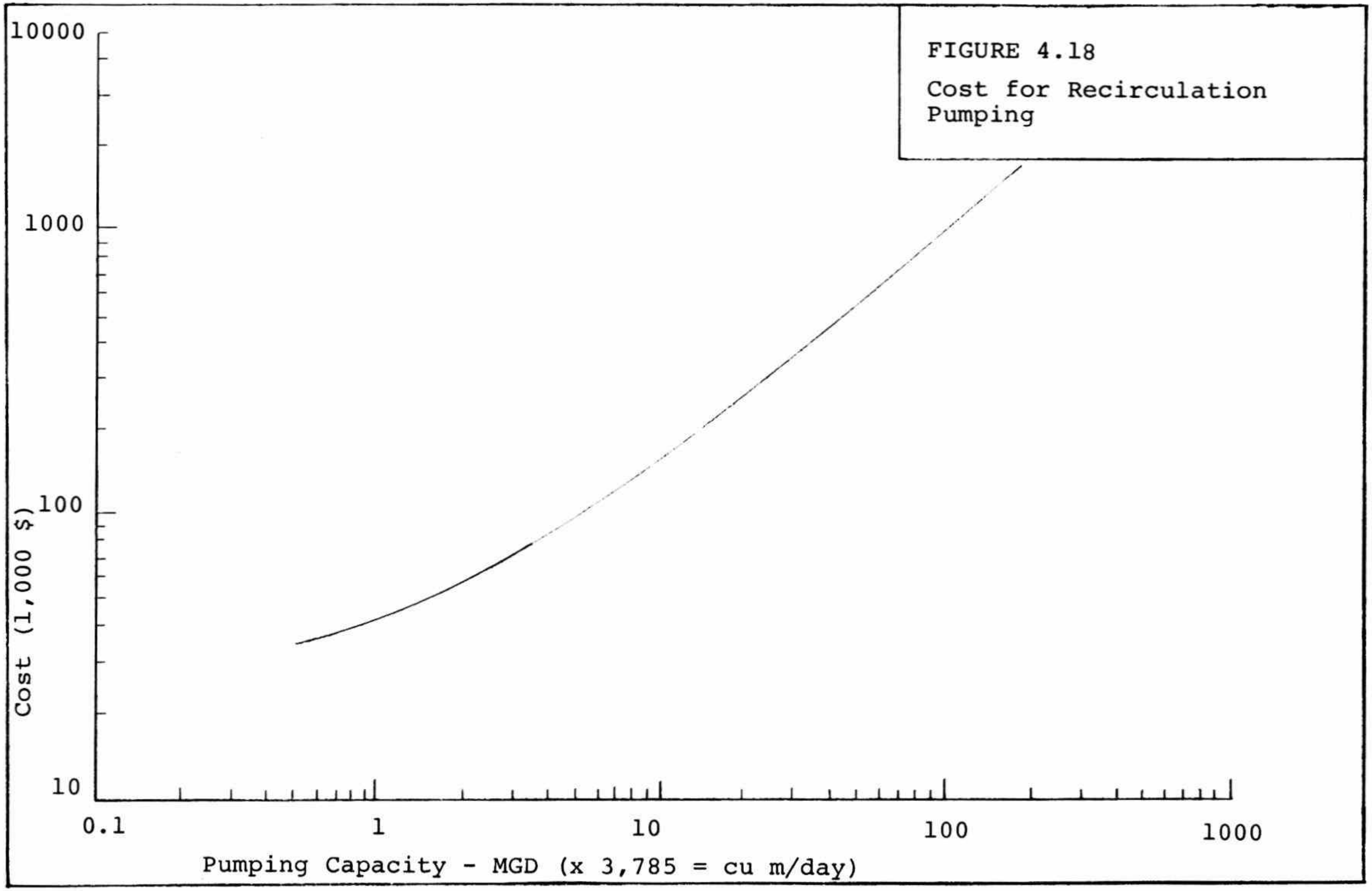


FIGURE 4.18
Cost for Recirculation Pumping

FIGURE 4.19
Average Daily Suspended Solids Concentration from Final Clarifiers

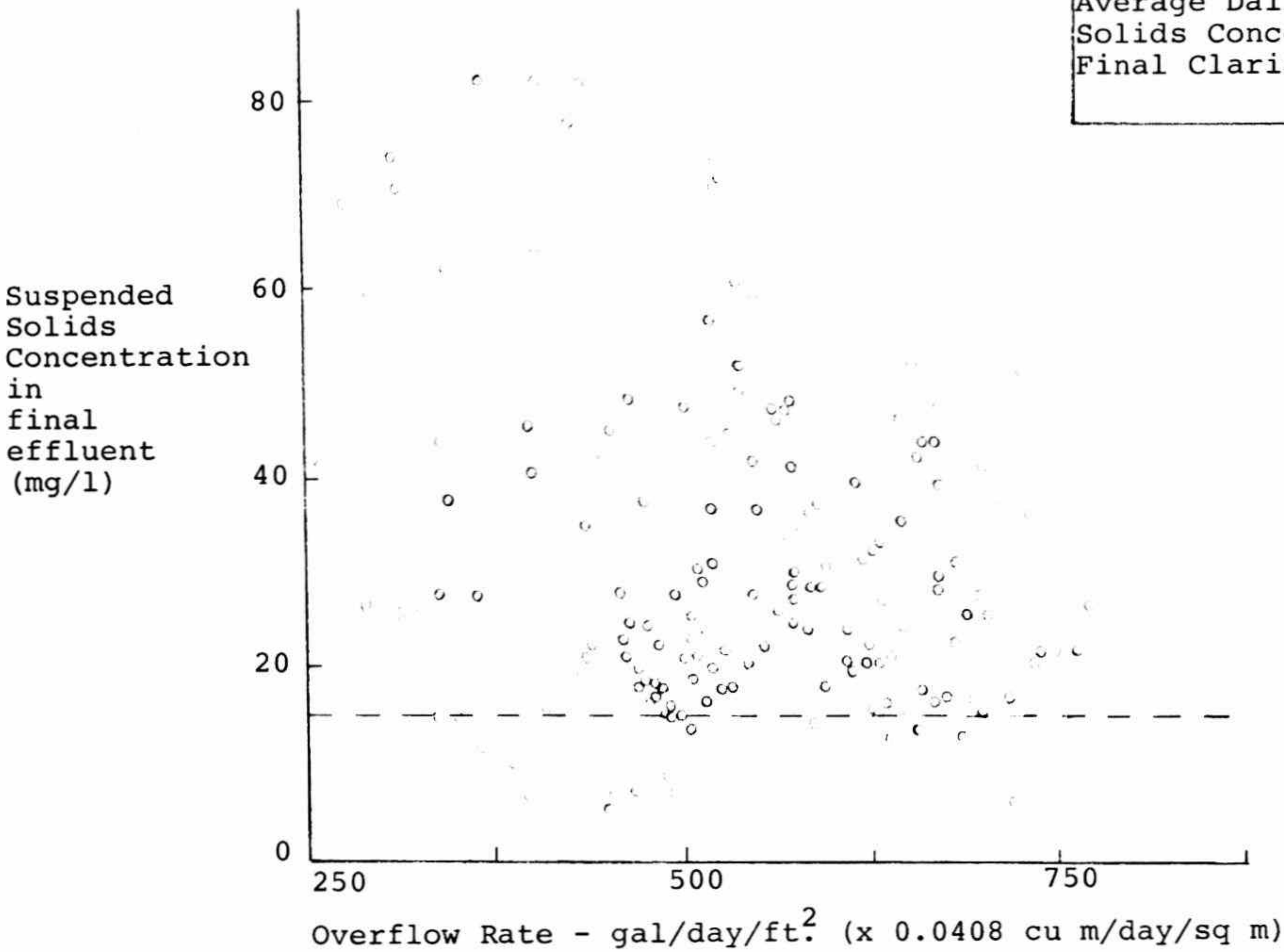
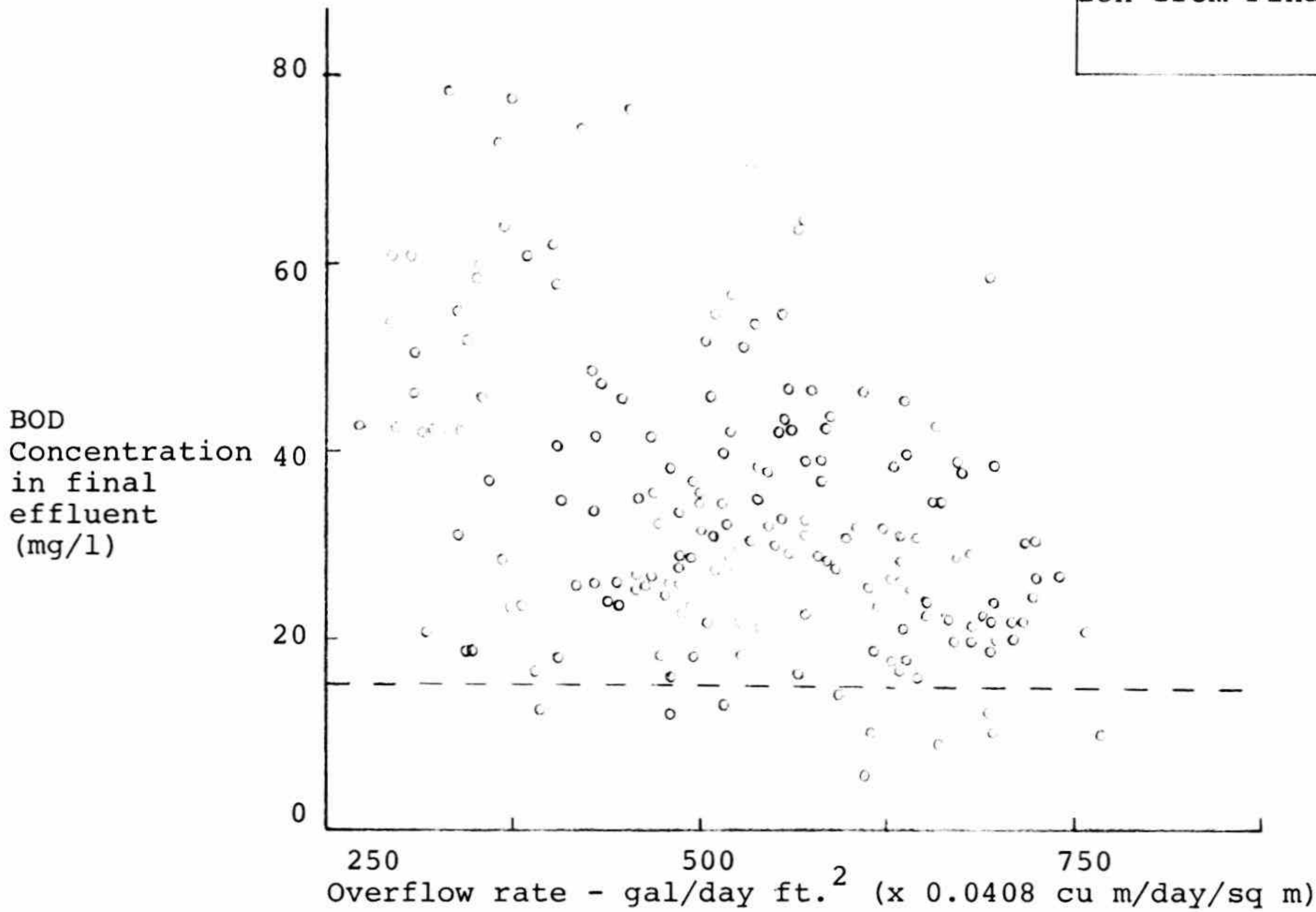


FIGURE 4.20
Average Daily BOD Concentration from Final Clarifiers



drawn at 15 mg/l, to represent a typical effluent quality requirement. The results indicate that during the year of operation under study the limits were not met for either BOD or suspended solids for most of the time. It must be noted that it was assumed that all clarifiers were in operation to calculate the overflow rates as information on clarifier operations was not provided on the operating records. Hence, some of the points shown in the plot may be shifted to the left as a result of using low overflow rates.

4.4.1 Cost of Existing Facilities

The final clarifier surface area required under fluctuating flow conditions was calculated from Equation 3.16.

$$A_F = \frac{f_b \; Qmax}{OR_F} \qquad 3.16$$

The maximum overflow rate selected for design purposes was 800 gal/day/ft.^{2} (32 cu m/day/sq m). As a rational basis for comparison with the equalized plant, actual flow data for the year of operation under study were used. This resulted in the following parameters

f_b = 1.28 - Figure 4.1

Qmax = 63 MGD (238,455 cu m/day) - Figure 4.2

The clarifier size can thus be calculated as

$$A_F = \frac{1.28 \times 63 \times 10^6}{800} = 101{,}000 \text{ sq ft}$$

(9,383 sq m)

Plant "A" has 12 final clarifiers. The cost of these clarifiers can be obtained from Figure 4.7.

Total Cost \$1,920,000

Each Clarifier - 8,500 sq ft (790 sq m)
- \$160,000

4.4.2 Cost of Equalized Facilities

For a plant with equalized flow conditions the design overflow rate, as discussed in Section 3, was selected as 600 gal/day/ft^2 (24 cu m/day/sq m). The daily peaking factor under equalized conditions is 1 thus the total final clarifier surface area required becomes

$$A_F(\text{equal}) = \frac{63 \times 10^6}{600} = 105,000 \text{ ft}^2 \text{ (9,754 sq m)}$$

Thus a larger final clarifier area would be required under equalized flow conditions. If the existing clarifiers in Plant "A" were used, the overflow rate for maximum flow conditions would equal

$$OR_{max} = \frac{63 \times 10^6}{101} = 625 \text{ gal/day/ft}^2 \text{ (25 cu m/day/sq m)}$$

This maximum overflow rate exceeds the design overflow rate by 4 percent and would occur only at peak flow. In addition, the fluctuating flow plant should be slightly larger than designed as it is based on a dry weather flow peaking factor instead of a wet weather flow peaking factor. Thus, for the purpose of this study it can be assumed that no change will occur in final clarifier size under equalized flow conditions. It is expected that further investigation into the effect of equalized flow on final clarification

will demonstrate that savings are possible under equalized flow conditions.

4.5 COST COMPARISON OF EQUALIZED FLOW AND VARYING FLOW PLANTS

The cost of equalization facilities and the savings which would be realized in other treatment units when a plant is designed for equalized flow conditions have been developed earlier in this section. These can be summarized as follows:

OUTLAY	Equalization basin		\$ 535,000
SAVING			\$ 677,500
	Primary clarifiers	\$65,000	
	Aeration basin	0	
	Air blower equipment	450,000	
	Return sludge pumps	162,000	
	Final clarifiers	0	
NET SAVING			\$ 142,500

This summary indicates that there would be a net capital cost saving by designing a plant for equalized flow conditions. It must be noted that the costs used in the calculations are average costs based on curves obtained from data of several plants. In practice, a specific number and size of units (clarifier, sludge pumps, air blowers, etc.) are selected. Because these units are generally available only in certain specific sizes, the design sizes and resultant costs may be higher or lower than those indicated.

The net saving in aeration equipment and return sludge pumping, which comprises the major portion of the saving, is based on design techniques not widely accepted, but due to current concern for the prevention of flow bypassing and

odour in activated sludge plants it is felt that these design practices will soon be more widely used and the resultant cost saving is justified. In addition, further study will likely show that cost savings in the design of the final clarifiers and aeration basins can be obtained.

All costing was based on the capital cost of equipment and in no way reflects upon the operating cost. By operating under constant flow conditions considerable operating cost savings can be obtained, for example, less horsepower is required for usage of the operation of air blower equipment.

5. DISCUSSION

The purpose of this study was to determine the effect of controlling waste variations on the operation of a sewage treatment plant.

To achieve this a preliminary methodology has been developed which will allow the comparison of capital cost estimates for municipal waste treatment facilities with and without facilities to equalize diurnal flow variations. The methodology was applied using information gathered from an operating sewage treatment plant.

A completely detailed approach in this first test of the methodology was not possible because of the shortage of suitable operating and research data. Furthermore, it was not intended to carry out a detailed analysis of the methodology using operating data from several plants but rather, to limit the comparison to one plant and to define areas where additional information is required.

The costs estimated for the comparison were conservative, i.e., where the information was insufficient to provide a realistic estimate of the facilities required in the treatment plant subjected to equalized sewage flows, values were chosen which would result in facilities that are larger than might be considered necessary. Operating costs were not included in the comparison. Even with this conservatism in approach, the study concluded that capital cost savings would be achieved when flow equalization was employed.

Specific areas of information deficiency where the use of conservative estimates were made, are;

a) PRIMARY CLARIFICATION

The plant data obtained for the efficiency of suspended solids removal versus flow possessed many anomalies and it was necessary to use a relationship based on the experience at many other waste treatment facilities. The effect of using this information is unknown as there was no way to correlate the source of data with information gathered from Plant "A". The efficiency plot for the primary tanks in Plant "A" may have been above or below the experience plot used.

Concerning the effect of diurnal flow variation on efficiency of solids removal, information gathered from the literature would implicitly include the effects of such variations. Although a factor was chosen, based on experience of comparing batch flow laboratory studies with actual plant data, to modify the information in the literature such that it represented an equalized diurnal flow it is felt that this can only be a first approximation to what might happen in actual practice.

b) AERATION

Probably one of the principal benefits of flow equalization, that of maintaining the biological processes under constant loading conditions, could not be estimated. It would be expected that parameters such as BOD removal rate and settleability of the activated sludge would improve as the degree of variation in the environment of the microorganisms was decreased. This would result in a decrease in the size of the aeration facilities and the final tanks. No allowance was made for this in the cost estimating.

c) FINAL TANKS

A penalty was applied to the final tanks on the equalized flow situation based on judgement. This resulted in the requirement for larger final tanks under the equalized flow condition than under a varying flow condition. With further information it is expected that this might not be the case.

d) FLOW BYPASSING

The methodology developed for comparing the sizes of equalized and varying flow plants was based on the principle of completely eliminating flow bypassing. The methodology for sizing units downstream of equalization facilities has not been expanded to handle situations where only partial elimination of flow bypassing occurs.

Flow bypassing from existing plants can be controlled by either expanding existing facilities or providing an equalization facility. Although this application has not been reviewed, it is expected that future studies will indicate that providing equalization facilities will present the least expensive alternative in addition to providing improved plant performance.

Several items which would be specific to a particular facility were not considered in the comparison. These are; the type and location of an equalization tank, the pumping requirements associated with equalization, and the degree of quality equalization achieved.

The conclusions drawn from the study were;

1. A methodology has been developed for sizing equalization

facilities to partially or fully equalize the flow to a waste treatment plant.

2. A preliminary methodology has been developed to assess the effect of equalization on the sizing and operation of downstream facilities. Insufficient data were available from either an operating plant or the literature on:

   i) the effect of diurnal variations on the efficiency of suspended solids removal in primary tanks;

   ii) the efficiency of suspended solids removal versus flow in primary tanks;

   iii) the effect on the biological processes of flow equalization with regard to BOD removal rates and settleability of the activated sludge;

   iv) the selection of a suitable overflow rate for the final tanks.

3. It was possible to demonstrate a capital cost saving by the addition of equalization facilities to a 40-50 MGD (15,140 to 18,924 cu m/day) municipal treatment plant. The savings demonstrated were significant and with consideration of the conservatism employed in the estimating procedure, it is felt that further study is warranted.

4. Although much of the necessary data must be obtained from pilot plant or full-scale plant work, it was concluded that further elaboration of the methodology is required so that satisfactory identification of the necessary information can be made.

6. RECOMMENDATIONS

Based on the foregoing conclusions it is recommended that:

a) The methodology should be expanded to include:

- the analysis of further operating data,
- the inclusion of operating cost comparisons in the methodology,
- the effect of partial equalization to the waste treatment facility.

b) After the further development of methodology and the more rigorous definition of information required, that pilot plant work be initiated to collect the necessary information.

c) The application of flow equalization to new plants, existing overloaded plants and the elimination or reduction of storm water flows be further investigated.

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APPENDIX A-1

Definition of Flow Terms

1.	Daily mean flow	The total volume of sewage influent to the waste treatment facility in a 24-hour period expressed as a uniform rate over the day, i.e. MGD. The daily mean flow can and will vary from day to day.
2.	Annual daily mean flow	The average value of the mean daily flows taken over a one year period. The annual mean daily flow can and will vary from year to year.
3.	Maximum daily mean flow	The maximum total volume of sewage influent to the waste treatment facility in any 24-hour period expressed as a uniform rate over the day for the period of record or estimated for the design year, i.e. MGD.
4.	Dry weather flow	Daily mean flow of sanitary sewage plus infiltration.
5.	Average dry weather flow	Annual daily mean flow of sanitary sewage plus infiltration.

APPENDIX A-2

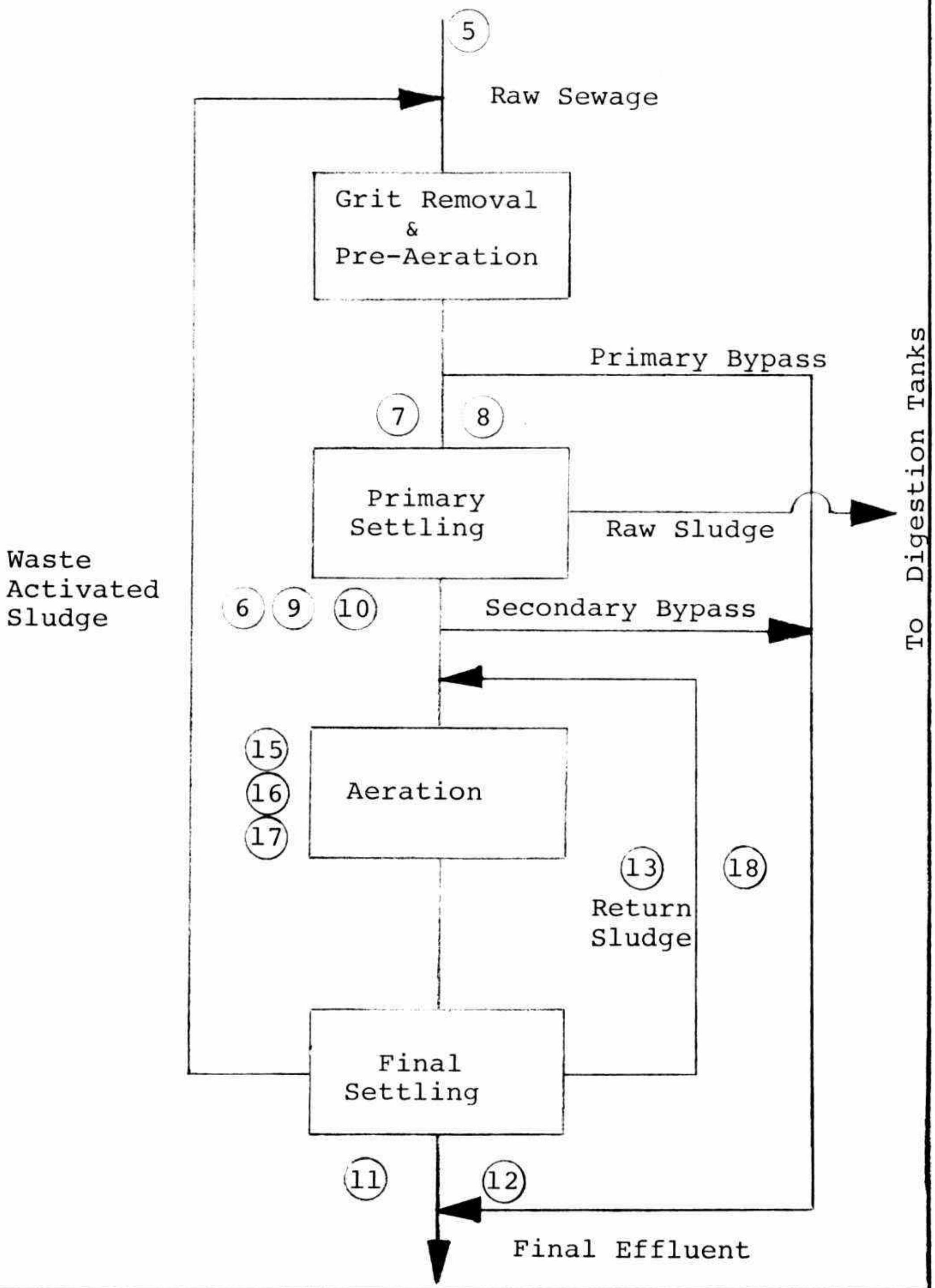
Nomenclature

A_F	Surface area of final clarifier
A_n	Trial selection of primary clarifier surface area
A_P	Surface area of primary clarifier
BOD_R	Biological oxygen demand removed in a unit
BOD_{IN}	Biological oxygen demand in the influent stream to a unit
BOD_{OUT}	Biological oxygen demand in the effluent stream from a unit
C_B	Saturation of oxygen in water at operating temperature and 1 atmosphere dry pressure
C_S	Concentration of solids in the waste activated sludge
DO	Dissolved oxygen level maintained in the aeration basin
DT	Detention Time
F/M	Organic loading rate
f_a	Maximum 12 hour flow peaking factor on mean day
f'_a	Maximum 12 hour flow peaking factor
f_b	Maximum 6 hour flow peaking factor
f_c	Maximum hour flow peaking factor
f_d	Flow peaking factor due to Daily Variations
f_e	Factor accounting for varying flow conditions in primary clarifiers
f_m	Ratio of hourly flow to mean daily flow
f_n	Factor accounting for non-idealized conditions in primary clarifiers
f_p	Primary clarifier flow peaking factor
f_s	Flow peaking factor due to seasonal variations

f_v	Scale-up factor from batch clarifiers to actual plant conditions
k	First order BOD removal rate constant
MLVSS	Mixed liquor volatile suspended solids
O_a	Actual oxygen requirement in activated sludge plant
O_s	Standard oxygen requirement in activated sludge plant
OR_F	Design overflow rate of final clarifier
OR_{FE}	Design overflow rate of final clarifier under equalized flow
OR_P	Design overflow rate of primary clarifier
P	Atmospheric pressure at treatment plant location
Q	Flow
Qm	Daily mean flow
Qmax	Maximum daily mean flow
Rm	Mean daily return sludge flow
Rmax	Maximum daily return sludge flow
Sa	Capacity of air equipment in English Units
Sam	Capacity of air equipment in Metric Units
T	Temperature of mixed liquor in aeration basin
V_A	Volume of aeration basin
V_{ES}	Volume of equalization basin in English Units
V_{ESM}	Volume of equalization basin in Metric Units
V_M	Fraction of mean daily flow requiring storage
V_P	Volume of primary clarifiers
α	Ratio of the oxygen transfer coefficient (K_La) of waste to that of clean water
β	Ratio of oxygen saturation in waste to that of clean water

APPENDIX A-3

Schematic Diagram



APPENDIX A-4

PLANT "A" SUMMARY SHEET
OPERATING DATA EXTRACTED FOR COMPUTER ANALYSIS

(Numbers Refer to Location of
Sampling Points As Indicated in Appendix A-3)

1. Day
2. Month
3. Year
4. Air Temperature
5. Sewage Temperature
6. Flow
7. Suspended Solids in Raw Sewage
8. BOD in Raw Sewage
9. Suspended Solids in Primary Effluent
10. BOD in Primary Effluent
11. Suspended Solids in Final Effluent
12. BOD in Final Effluent
13. Percent Solids in Return Sludge
14. Daily Volume of Waste Activated Sludge
15. Average Sludge Volume Index
16. Average Dissolved Oxygen Residual in Aeration Basin
17. Average Mixed Liquor Suspended Solids in Aeration Basin
18. Daily Volume of Return Sludge